Kansas Citys, Missouri and Kansas
Flood Damage Reduction Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Interim Feasibility Report

Chapter A-4

GEOTECHNICAL ANALYSIS
EXISTING CONDITIONS

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A-4.1 INTRODUCTION

The purpose of this portion of the study is to evaluate the existing performance of Kansas Citys' (Missouri and Kansas) flood protection systems. This was done based on underseepage and landward side slope stability analyses. The evaluations were done in accordance with the USACE Engineering Technical Letter (ETL) 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." The results of this phase of the study were used to determine the benefits attributed to potential levee raises or possible levee strengthening. They also helped to guide the additional geotechnical analyses found in subsequent chapters of this appendix.

A-4.2 SOURCES OF INFORMATION

The primary sources of information for this geotechnical analysis include the documents listed in the References section of this chapter.

A-4.3 DESCRIPTION OF THE LEVEE UNITS

A-4.3.1 Argentine Unit

The Argentine Unit is located in Wyandotte County, Kansas on the right bank of the Kansas River between approximate Kansas RM 10.1 and RM 4.75. The unit begins at the Santa Fe Railroad embankment upstream from the Turner Bridge, Station 0+00, and extends downstream to Station 288+30, immediately upstream from the 12th Street Bridge. It consists of a system of levees, floodwalls, stoplog gaps, sandbag gaps, riprap and levee toe protection, and surfaced levee crown and ramps.

The Argentine Unit was constructed for flood control of the Kansas River. It diverts flow upriver along the hillside to Barber Creek. The Santa Fe ditch is located just south of the Argentine Unit levee from approximate Stations 0+00 to 28+06, where the ditch then joins Barber Creek. Barber Creek has been excavated to provide a uniform stable channel to the Kansas River.

The first levee section for the Argentine Unit begins at Station 0+00 and is interrupted with the first floodwall at Station 251+65. The floodwall continues to Station 253+92 and protects the Argentine Boulevard Pumping Plant. The second levee section then resumes until Station 276+70. The second floodwall begins at Station 276+70 and extends east, or downstream, along the Santa Fe railroad tracks to Station 287+92. The Argentine Unit ends with a stoplog gap at Station 288+57.

A-4.3.2 Armourdale Unit

The Armourdale Unit is located in Wyandotte County, Kansas, along the left bank of the Kansas River from RM 6.4 to RM 0.3, near the junction of the Kansas River with the Missouri River. The flood protection unit consists of a system of levees, floodwalls, riprap and toe protection on riverward slopes of levees, toe drains along the concrete floodwalls, surfaced levee crown, ramps and turnouts, seeded landside slopes of levees, sandbag and stoplog gaps, drainage structures, relief wells, and pumping plants. The drainage area property consists of approximately 2,000 acres, which includes industrial property, railroad yards, businesses, and residences.

The Armourdale Unit consists of four sections of levee divided by floodwalls and closure gaps. The first levee section begins at Station 0+00 U.E. (Upper End) and continues to Station 20+09 U.E., which is equal to Armourdale stationing of 9+71. The levee continues to Station 60+30 where a floodwall begins. The first section of floodwall begins at Station 60+30 and ends at Station 77+78. The second section of levee begins at Station 77+78 and goes to Station 246+90, where the second floodwall begins and runs to Station 250+50. The next section of levee begins at Station 250+50 and extends to Station 257+65. The last section of floodwall begins at Station 257+65 and ends at Station 302+58. The final section of levee begins at Station 302+58 and ends at Station 354+14, which is equal to Station 61+00 L.E. (Lower End). The levee sections were constructed with a crown width of 10 feet. The levee crown, turnouts, and ramps are protected with a 5-inch thick crushed rock surfacing.

A-4.3.3 Birmingham Unit

The Birmingham Unit is located in Clay County, Missouri, on the left bank of the Missouri river, approximately 12.4 miles downstream from the mouth of the Kansas River, extending downstream from Missouri RM 371.6 to RM 361.5 (1941 mileage adjusted). Big Shoal Creek flows approximately parallel to the northern edge of the unit entering the Missouri River by way of the old Liberty Bend channel. The flood protection works in the Birmingham Unit consists of an earth-fill levee, a small section of floodwall, riprap slope protection on a section of riverward slope, crushed rock surfacing of levee crown, ramps, drainage structures, sandbag and stoplog gaps, and underseepage control and stability berms. The protected area consists of approximately 5,000 acres of agricultural land and the small community of Birmingham, Missouri.

The Birmingham Unit consists of an earth-fill levee, which begins at the bluff, southeast of Randolph, Missouri at Station -24+30. It then extends downstream along the left bank of the Missouri River to Station 310+00. Then the levee extends north and west along the left bank of the old Liberty Bend channel to the Wabash Railroad, then upstream along the right bank of the Big Shoal Creek where it finally ties into high ground at the Liberty Road (Station 558+35). Included in the levee section is a small piece of floodwall between the stoplog and sandbag gaps from Station 555+53 to 557+90.

A-4.3.4 Central Industrial District-Kansas Unit (CID-KS)

The Central Industrial District – Kansas flood protection unit is located in Wyandotte County, Kansas, and extends from the Kansas/Missouri state line along the right bank of the Missouri River to the mouth of the Kansas River. It then continues upstream along the right bank of the Kansas River to RM 3.4. The unit consists of a system of levees and floodwalls, stoplog gaps, sandbag gaps, pumping plants, riprap and levee toe protection, and surfaced levee crown and ramps. The greater portion of the area to be protected consists of 360 highly industrialized areas. These areas are occupied largely by stockyards, railroads, wholesale houses, and manufacturing plants. The area of interior drainage also includes 352 acres along the bluffs to the south and east.

Both the Kansas and the Missouri Rivers affect the CID – Kansas Section. The first levee section begins at Station -6+36 (83+01 CID-MO) on the Kansas/Missouri state line. It extends upstream along the right bank of the Missouri River to the mouth of the Kansas River, then upstream along the right bank of the Kansas River to the James Street viaduct at Station 26+73.

The first section of floodwall begins at Station 26+73, near the James Street Bridge, and extends upstream along the right bank of the Kansas River to Station 40+31, near the Kansas City Southern Railroad Bridge.

The next section of levee begins at Station 40+92 and extends upstream along the right bank of the Kansas River to a junction with a floodwall from Stations 74+36 to 77+28. This floodwall is divided by the sandbag gap opening for the Union Pacific Railroad tracks. The levee then continues from Station 77+28 to Station 102+73.

The last main section of floodwall begins at Station 102+73, near the Chicago, Rock Island and Pacific Railroad Bridge, and extends upstream along the right bank of the Kansas River to the end of the project, Station 166+25, near the Seventh Street Bridge.

A-4.3.5 Central Industrial District-Missouri Unit (CID-MO)

The Central Industrial District-Missouri Unit is located in Kansas City, Jackson County, Missouri. The unit extends along the right bank of the Missouri River, upstream from the Grand Avenue Viaduct (RM 365.7) to the Kansas-Missouri state line (RM 367.2). The unit consists of a system of levees, floodwalls, drainage structures, a levee drainage system, sandbag and stoplog gaps, toe and bank protection, and slope protection on the riverward slope.

The first floodwall section of the CID-MO Unit, begins at the Grand Avenue Viaduct, Station 0+00, and extends westerly to where it connects with the permanent levee at Station 78+12. From Station 78+00 to Station 78+12 there is 12 feet of overlap with the levee.

The first section of the levee begins at Station 78+00 and extends westward to the Missouri-Kansas state line at Station 83+01.

A-4.3.6 East Bottoms Unit

The East Bottoms Unit is located in Kansas City, Jackson County, Missouri. The unit extends downstream along the right bank of the Missouri River from the Armour-Swift-Burlington (A.S.B.) Bridge, RM 365.6, to the mouth of the Big Blue River, RM 357.7. It then extends upstream along the left bank of the Big Blue River to the Missouri Pacific Railroad Embankment. The unit consists of a system of levees, floodwalls, stoplog gaps, collector pipes, relief wells, and pump plants.

The first section of levee begins at the A.S.B. Bridge at Station 0+00 and continues to Station 57+26. This portion of the levee contains the Riverfront Park and the Isle of Capri Casino improvements. The first floodwall, at the KCP&L Hawthorn Station, begins at Station 57+14 and extends east, or downstream, to Station 74+56. This wall includes a stoplog gap at Station 65+13.

The levee continues from Station 74+56 to Station 501+00 at the end of the system. A stoplog gap is incorporated into a small floodwall from Stations 472+55 to 473+35 and at Stations 475+08 to 478+78.

A-4.3.7 Fairfax-Jersey Creek Unit

The Fairfax-Jersey Creek Unit is located in Wyandotte County, Kansas, on the left bank of the Kansas River from the Missouri Pacific Railroad Bridge (Kansas RM 0.3) downstream to the mouth of the Kansas River, and along the right bank of the Missouri River from Missouri RM 367.5 to RM 373.9 (1960 adjusted mileage). The flood protection facilities consist of levees, floodwalls, stoplog and sandbag gaps, riprap and levee toe protection, surfaced levee crown and ramps, drainage structures, pressure relief wells and levee drainage system, and the Jersey Creek sewer structure and shutter gate and pumping plants.

The first levee section begins at Station -5+59 and extends upstream to Station 2+58. The first section of floodwall begins at Station 2+58 and extends upstream to Station 28+51. The next section of the levee begins at Station 28+51 and continues to Station 287+98 with the floodwall on top of the levee for the last 12'. The second and last section of the aforementioned floodwall is from Station 287+86 to Station 302+32 with the first 12' of the floodwall on top of the preceding levee. The final section of levee then begins from Station 302+32 to Station 313+72.

A-4.3.8 North Kansas City-Airport Unit

The North Kansas City-Airport Unit is located in Kansas City, Clay County, Missouri. The unit consists of a system of levees, floodwall, and appurtenances along the left bank of the Missouri River with the levee extending downstream from Station 70+40 to Station 201+94. This corresponds to Missouri RM 369.6, 1960 adjusted to Missouri RM 366.2, 1960 adjusted.

A retaining wall precedes, by 154', the only section of floodwall. The floodwall begins at Station 203+48 and extends to Station 208+82 with the last 12 feet extending onto the second section of the levee. This levee begins at Station 208+70 and extends to the end of the North Kansas City- Airport Unit at Station 210+40.

A-4.3.9 North Kansas City-Lower Unit

The North Kansas City-Lower Unit is located in North Kansas City, Clay County, Missouri. The unit consists of a system of levees, floodwalls, riprap slope protection, Rock Creek channel relocation, underseepage collector system, underseepage control berms, pumping plants, bridge and approach alterations to Hannibal and A.S.B. Bridges, and emergency railroad crossing. The unit extends downstream along the left bank of the Missouri River from the bluff just north of the Kansas City, Missouri, Waterworks Intake (Station 0+00 to Station 70+40), and then begins again downstream from the Hannibal Bridge at Station 210+40. The levee continues along the left bank of the Missouri River, approximate Station 359+60, along the hillside ditch west of Cherry Street to Station 469+17.

A-4.4 SITE CONDITIONS

A-4.4.1 General Geology of the Region (Missouri River)

The units are near the southern edge of the Dissected Till Plains section of the Central Lowlands Physiographic Province. The southern limit of glaciation in Missouri is generally considered to be just south of the Missouri River. During the Pleistocene, both the Nebraskan and Kansas glaciation crossed Platte County. The topography consists mainly of flat-lying alluvial sediments of the Missouri River floodplain, bounded by rolling hills comprising the valley walls. Maximum relief in the area is about 170 feet. The Missouri River alluvium generally ranges from approximately 110 to 130 feet in thickness, with the exception of buried stream channels that may extend into the Marmaton Group. All of the Missouri alluvium lies on shales and siltstones in the Pleasanton Group of the late Pennsylvanian System. The valley walls are composed of alternating layer of shales and limestone of the Kansas City Group. Drainage is by means of a maturely developed dendritic pattern except where it has been altered by human activity.

A-4.4.2 General Geology of the Region (Kansas River)

The Kansas River Valley, near its mouth, is cut into Pennsylvanian bedrock of the Missourian Series. The oldest bedrock exposed is in the Bethany Falls Limestone member of the Swope Limestone formation, Kansas City Group. Bedrock of the Missourian Series is characterized by numerous limestone beds separated by clayey to somewhat sandy shale. The bedrock is generally overlain by much younger unconsolidated materials consisting of glacial drift, loess of the Pleistocene age, alluvium deposits and isolated remnants of till of Kansas stage ice sheet occurring on the hilltops. The Kansas River is near the southern edge of Kansas glaciation. Wind blown deposits of silt (loess), form an irregular deposit covering much of the eastern part of Wyandotte County. Alluvium, ranging from clay and silt to sand and gravel, occurs in the Kansas River Valley. Much of this alluvium is probably of glacial origin, having been deposited as glacial outwash from the melting ice sheets.

A-4.4.3 Subsurface Conditions

Assessments of the subsurface conditions along the various units were derived from a variety of sources consisting of Record Drawings, Design Memoranda, and borings made at selected sites during the feasibility study. Typical subsurface blanket conditions generally consist of silts, sandy clays and lean clays of variable thickness ranging from 0 to 30 feet.

A-4.5 LEVEE DESIGN FEATURES

A-4.5.1 Basic Levee Sections

The basic levee sections were constructed with a 10 to 15 foot crown width, with side slopes ranging from 3:1 to 4:1 horizontal to vertical riverside and landside levee slopes. Underseepage and stability berms were added when necessary in certain reaches. The levee embankment consists of compacted earthen material placed in random and impervious zones. Rock slope protection was provided on the riverside slopes where needed and around inlets and outlets of drainage structures. All other slope surfaces are protected by established grasses. The levee crown, turnouts, and ramps are surfaced with 6 inches of aggregate surfacing.

A-4.5.2 Seepage Control Measures

Seepage control measures consist of underseepage berms, relief wells, toe collectors and area fill where necessary. Typical locations of existing underseepage controls are located where the natural blanket is thin and where there is adequate room in localized areas.

A-4.5.3 Stability Berms

Levee sections were designed to provide a minimum factor of safety of 1.25 for the riverward submerged toe case, and 1.5 for the steady seepage case on the landside. Typically, stability berms were used for levee sections over 10 feet. For the existing soil conditions, this appears to be the limiting height, or spring point between the elevations of top of levee and top of berms.

A-4.6 ASSESSMENT OF LEVEE INTEGRITY

The current levee systems in the Kansas Citys flood protection area at this time are rated good to very good. Since the 1951 flood event, most, if not all, of the Kansas Citys flood protection systems have been upgraded in response to damage or problems experienced in 1951. After the upgrades to the system, the largest flood event experienced by the units in the Kansas City area was the 1993 flood. In the existing conditions phase of the study, the 1993 river levels were used as a baseline to evaluate the performance of the levee units. During the 1993 flood event, several localized problem areas were experienced, although none of the problems resulted in serious inundation of any of the flood protection units.

The Fairfax-Jersey Creek Unit experienced several localized failures in its system due to seepage at gatewell structures and pipe connections. Problems also were encountered with collector systems at the base of floodwalls due to the removal of riser pipes. In 1968, one reach of the river bank sheet pile support collapsed. The flood protection was not affected as river stages were low, and the sheet pile was repaired. None of the problems encountered resulted in serious interior flooding. After the 1993 event, all problems within this system were repaired where necessary and deficiencies in the collector systems were upgraded. More recently, the levee district provided an independent evaluation of the section of retaining sheet pile wall from Station 23+30.6 to 29+98.9. This report identified the failure of the tieback connections and extensive rusting of the existing retaining wall structure. The retaining wall structure provides stability of the foreshore bank for the existing levee with I-wall flood protection.

Problems in the North Kansas City Levee District's Lower Unit were reported in the Harlem area that is located from approximately Station 210+00 to 240+00. The National Starch area, extending from Station 240+00 to 275+00, also exhibited problems. In 1993, the high water elevation was nearly 4 feet from the top of the levee. At that time, reports indicated that underseepage pressures were causing noticeable pumping of the ground behind the levee. Although no failures occurred in this area, it was evident from field reports in 1993 that if water levels had reached higher elevations major problems could have developed, or even failure of the levee system.

The East Bottoms Levee Unit also had reports of underseepage problems in the reach approximately from Station 380+00 to 401+00. Reports from various sources indicate that, during the 1993 flood event, large sand boils developed in this area at the high water elevation. At this location, the river elevation was approximately 3.5 feet from the top of levee. During that time, no major failures occurred. At higher river levels, however, the potential for consequences that are more serious could exist. Both of these locations were chosen for strengthening in the future conditions analysis.

A-4.7 PROBABILISTIC THEORY

A-4.7.1 Probabilistic Parameters

Several parameters are commonly used to describe probability distributions such as the one shown in Exhibit A-4.1. Probably the most common of these is the mean or expected value. The expected value of a continuous random variable *X* (a variable that can take on any value within some continuous range) with some distribution *f(x)* is defined as:

$$\mu_X = \int_{-\infty}^{\infty} x_i \, f_X(x) \; dx \qquad \text{Equation A-4.1}$$

where μ_X is the mean value of the random variable *X*, x_i is a particular value of the random variable *X* and $f_X(x)$ is the frequency of occurrence of the random variable *X*. Stated in words, the expected value of a random variable is the weighted average of the values of the random variable with the weighting being the frequency of occurrence of the value. For a set of discrete measurements of a random variable, the mean value is computed as:

$$\mu_X = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \text{Equation A-4.2}$$

The variance of the random variable *X*, Var[X], is a measure of the spread, or variability of the random variable about the mean. The variance is computed as:

$$\text{Var[X]} = \int_{-\infty}^{\infty} (x_i - \mu_X)^2 \, f_X(x) \; dx \qquad \text{Equation A-4.3}$$

For a set of discrete measurements of a random variable *X*, the variance is computed as:

$$\text{Var}[X] = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N} \qquad \text{Equation A-4.4}$$

If the number of observations N is a relatively small set of an entire population, an unbiased estimate of the variance can be given as:

$$\mathrm{Var}[X] = \sigma_X^2 = \frac{\sum_{i=1}^{N}(x_i - \mu_X)^2}{N-1} \qquad \text{Equation A-4.5}$$

The standard deviation, σ_X, is also a measure of the distribution of the random variable about the expected value and is the square root of the variance:

$$\sigma_X = \sqrt{Var[X]} \qquad \text{Equation A-4.6}$$

The coefficient of variation, COV, is a convenient dimensionless parameter used to express the uncertainty or variability of a random variable and is computed as:

$$\mathrm{COV} = \frac{\sigma_X}{\mu_X} \qquad \text{Equation A-4.7}$$

The coefficient of variation is useful because it expresses the variability of a random variable normalized with respect to the mean of the random variable. The expected value, standard deviation and coefficient of variation are inter-related; therefore, the third can be determined by knowing any two of the parameters.

A-4.7.2 Probability Distributions. Many forms of probability distribution are available that can be used to represent the variability and uncertainty. However, based on previous work (Kitch, 1994) the normal and log-normal distributions are by far the most commonly use for risk based analyses.

The normal distribution is the most widely used distribution in the description of statistical phenomenon. The probability density function for a normally distributed random variable is expressed as:

$$f_X(x) = \frac{1}{\sigma_X\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu_X}{\sigma_X}\right)^2\right]dx \qquad \text{Equation A-4.8}$$

where $f_X(x)$ is the relative frequency of the random variable X and is not a probability, but a representation of the distribution of probability that a particular random variable may lie within some stated interval. As shown in Exhibit A-4.1, the normal distribution has a bell shape with upper and lower limits of positive and negative infinity.

Another distribution that has been proven useful for reliability-based analysis in geotechnical engineering is the log-normal distribution shown in Exhibit A-4.2. In the log-normal distribution, it is assumed that the natural logarithm of a random variable X is normally distributed. As shown in Exhibit A-4.2, the log-normal distribution is positively skewed towards the lower values. However, it has the distinct advantage that the probability of the random variable cannot be less than zero. The log-normal distribution is therefore useful for representing parameters that cannot take on negative values (e.g. factors of safety and hydraulic gradient).

If a random variable X is log-normally distributed, the *ln X* is normally distributed. The probability density function can therefore be expressed as:

$$f_X(x) = \frac{1}{x\sigma_{\ln X}\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln x - E[\ln X]}{\sigma_{\ln X}}\right)^2\right] dx \qquad \text{Equation A-4.9}$$

where $\sigma_{\ln X} = \sqrt{Var[\ln X]}$.

A-4.7.3 Probabilistic Measure of Stability for Slopes

In reliability-based analysis of slopes, several of the input parameters are generally considered to vary according to some form of distribution as described in the previous section. These variable parameters are then used as input into a series of stability analyses to obtain the overall distribution of the performance function. The performance function is used to report the stability of the slope. The performance function used throughout this study for slope stability is the factor of safety.

A hypothetical distribution of the factor of safety that could result from analyses using probabilistic parameters is shown in Exhibit A-4.3. As shown in the figure, the distribution indicates that the actual factor of safety may take on a range of possible values, ranging from well below the limiting value of FS = 1.0 to well above the limiting value. While knowledge of the complete distribution of the factor of safety is useful, it is the relative frequency of factors of safety less than the limiting value that are of primary importance (FS ≤ 1.0 => Failure). Three different probabilistic parameters are typically used to represent this relative frequency.

The probability of failure of a system is the area under the probability density function shown as the shaded area in Exhibit A-4.3. For the log-normal function, this would be from the boundaries (0 ≤ FS ≤ 1). In mathematical terms it can be expressed as:

$$P_f = \int_0^1 f_X(x)\, dx \qquad \text{Equation A-4.10}$$

where $f_X(x)$ is the probability density function expressed in Equation A-4.8.

The reliability of a system is conversely the area under the probability density function bounded by the limiting value and positive infinity. In Figure A-4.3, it is represented by the non-shaded area under the curve. For a log-normal distribution, the boundaries would be (1< FS ≤ +∞). Since the total probability for all possible values of the random variable is 1.0, the probability of failure, P_f, and the reliability, denoted as R, are related by:

$$P_f = 1\text{-}R \qquad \text{Equation A-4.11}$$

Based on the assumption that the factor of safety is log-normally distributed, the natural log of the factor of safety will be normally distributed. In this case, the boundaries for the probability of failure would be ($-\infty <$ lnFS ≤ 0). Under this assumption, the probability curve and its probabilistic parameters would be represented in Exhibit A-4.4 with the probability of failure in the shaded area.

The reliability index, β, is a gage of the reliability of a system that takes into account technicalities of the procedure and the uncertainties introduced by random input variables. The reliability index gives a measure of comparative reliability for a system, thereby making it unnecessary to calculate or determine the actual probability distribution. It is defined using the probabilistic terms of standard deviation and the expected value (mean) of the performance function. Graphically, the reliability index multiplied by the standard deviation is equal to the distance from the expected value (mean) to the limiting state as shown in Exhibit A-4.3. For a log-normal distribution, the reliability index is computed as:

$$\beta = \frac{\ln\left[E[FS] \Big/ \sqrt{1 + COV[FS]^2} \right]}{\sqrt{\ln\left(1 + COV[FS]^2\right)}} \qquad \text{Equation A-4.12}$$

where β is the reliability index, *E[FS]* is the expected value (mean) of the factor of safety, and *COV[FS]* is the coefficient of variation of the factor of safety.

A-4.7.4 Probabilistic Measure of Stability for Underseepage Using Flow Nets

Where excess head at the top of the sand landward of the levee is zero, the evaluation of piping must be determined using a flow net as stated in EM 1110-2-1913 "Design and Construction of Levees" (30 April 2000). Using this type of analyses, the probabilistic measures discussed in the previous section were assumed to apply. The factor of safety is calculated as:

$$\text{FS} = \frac{i_f * n_e * L}{H} \qquad \text{Equation A-4.13}$$

where i_f is the failure gradient, n_e is the number of equipotential drops, L is the length from the last equipotential line to the landward side ground surface, and H is the height of the driving head above the tailwater.

A-4.7.5 Probabilistic Measure of Stability for Underseepage Without Flow Nets

When the excess head at the ground surface on the landward side of the levee toe is greater than zero and the blanket material is thicker than one-fourth the levee height, the probability of failure can be calculated using the method described in ETL 1110-2-556.

Using this method, the exit gradient (i) is assumed to be a log-normally distributed random variable with probabilistic moments E[i] and σ_i. Based on this assumption, the equivalent normally distributed random variable has moments E[ln i] and

$\sigma_{\ln i}$. The limit state for the underseepage would then be natural log of the failure gradient (i_f) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation A-4.14}$$

The probability of the ln i being greater than the ln i_f is determined by using the standard normalized variate (z), which is also analogous to the reliability index β. The standard normalized variate is calculated as:

$$z = \beta = \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\dfrac{i_f * \sqrt{1+COV^2}}{E[i]}\right]}{\sqrt{\ln(1+COV^2)}} \qquad \text{Equation A-4.15}$$

where, *E[i]* is the expected value (mean) of the hydraulic gradient and *COV[FS]* is the coefficient of variation of the hydraulic gradient. Figure A-4.5 shows a graphical representation of the probabilistic parameters for the underseepage analysis with the probability of failure in the shaded area.

A-4.7.6 Taylor Series Approximation Method for Determining Risk and Uncertainty Analysis

As described in the previous sections, the probability of failure can be computed if the expected value (mean) and variance of the distribution are known. Numerous methods are available for computing the probability of failure for reliability-based analyses, including first order second moment methods (FOSM), the point estimate method, the Hasofer-Lind method, and Monte Carlo simulations (Baecher & Christian 2000). While all of these methods can be used, the most commonly used method to date in geotechnical applications is the Taylor Series Approximation of the FOSM method (USACE, 1999). The basis of the Taylor series method is that it uses the first two linear terms on the Taylor series expansion of the performance function to determine the probabilistic measures of performance. As such, the method is exact for linear performance functions and is approximated for higher order functions. While this method is approximate from a strictly probabilistic point of view, it has the significant advantage of being relatively simple to implement.

For a function (Y) of random independent variables (X_1, X_2, . . .X_n) of the form

$$Y = g(X_1, X_2, \ldots X_n) \qquad \text{Equation A-4.16}$$

the expected value (mean) of Y can be found by evaluating the function at the expected values (mean) of the random variables. In the slope stability analysis application, the function Y is chosen to be the factor of safety and the random variables are the input parameters that are chosen as probabilistic. The expected value of the factor of safety is therefore computed directly from the expected values (mean) of the random variables.

Stated in mathematical form, this is:

$$E[FS] = FS(E[\bar{\phi}_{\text{foundation}}], E[\bar{\phi}_{\text{blanket}}], E[\bar{\phi}_{\text{embankment}}]) \qquad \text{Equation A-4.17}$$

where E[FS] is the expected value (mean) of the factor of safety and $E[\bar{\phi}_{\text{foundation}}]$, $E[\bar{\phi}_{\text{blanket}}]$, and $E[\bar{\phi}_{\text{embankment}}]$ are the expected values (mean) of the random variables.

The Taylor Series approximation for the variance of the factor of safety can be expressed as:

$$\text{Var[FS]} = \sum\left[\left(\frac{\partial FS}{\partial X_i}\right)^2 Var[X_i]\right] \qquad \text{Equation A-4.18}$$

where X_i represents a value of the i[th] random variable for the stability analysis, $Var[X_i]$ is the variance of that random variable, and $\frac{\partial FS}{\partial X_i}$ is the partial derivative of the distribution of the factor of safety evaluated at the expansion point. Noting that the $Var[X] = \sigma_X^2$ and approximating the partial derivative with a difference form, Equation A-4.18 becomes:

$$\text{Var[FS]} = = \sum\left(\left[\frac{\Delta FS}{\Delta X_i}\right]^2 \sigma_i^2\right) \qquad \text{Equation A-4.19}$$

where σ_i is the standard deviation of the i[th] random variable and $\frac{\Delta FS}{\Delta X_i}$ is the approximated partial derivative. It has become common to evaluate the partial derivative $\frac{\Delta FS}{\Delta X_i}$ at the expected value (mean) plus one standard deviation and at the expected value (mean) minus one standard deviation as shown in Exhibit A-4.6 so that $\Delta X_i = 2\sigma_i$. Making this simplification, the expression for the variance becomes:

$$\text{Var[FS]} = \sum\left(\frac{FS(E[FS]+\sigma_{FS}) - FS(E[FS]-\sigma_{FS})}{2}\right)^2 \qquad \text{Equation A-4.20}$$

where $FS(E[FS] + \sigma_{FS})$ is the factor of safety calculated at the expected value plus one standard deviation and $FS(E[FS] - \sigma_{FS})$ is the factor of safety calculated at the expected value minus one standard deviation. Noting that the $\sqrt{Var} = \sigma$, the equation for the standard deviation for the factor of safety will become:

$$\sigma_{FS} = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \ldots\ldots + \left(\frac{\Delta FS_n}{2}\right)^2} \qquad \text{Equation A-4.21}$$

where σ_{FS} is the standard deviation of the factor of safety and ΔFS is the difference between the factors of safety calculated at the expected value plus and minus one standard deviation for each of the random variables.

The discussion above describes how the factor of safety was evaluated as the limit state function. The exact same procedure can also be used with the critical hydraulic gradient as the limit state with different input parameters applicable to the underseepage analysis.

Once the standard deviation and expected value for the factor of safety are known, the coefficient of variation COV for the factor of safety may be calculated and then used in Equation A-4.12 to compute the reliability index. Given the reliability index, β, the probability of failure is calculated using the built-in function NORMSDIST in Microsoft Excel. This function uses the reliability index as the argument allowing for the probability of failure to be computed as:

$$P_f = 1 - \text{NORMSDIST}(\beta) \qquad \text{Equation A-4.22}$$

A-4.8 UNCERTAINTY ANALYSES

A-4.8.1 General

Risk-based analyses for the flood-damage reduction studies of Kansas Citys' Flood Protection Systems were performed as part of the existing conditions portion of this appendix. In these risk analyses, geotechnical uncertainties were assessed by developing probability distributions for the blanket thickness and soil material properties for typical levee sections representative of each levee system.

Geotechnical failures in this study are defined as failure of the embankment slope resulting in water from the river flowing to the landside areas of the levee, resulting in economic damages to the interior. Geotechnical failure may occur when river stages reach elevations at or below the top of levee. Within this range, geotechnical failure modes considered were excessive underseepage leading to a piping condition and slope failure on the landside of the levee under steady state seepage conditions.

In order to present the most accurate probabilities possible, certain conditions had to be met that were representative of observations made during the flood event of 1993. The main condition was that, if there was no evidence of problems at the 1993 water surface elevation, the probabilities of failure should not be more than 10 percent at that same water surface elevation. If there were definite problems at the 1993 river elevation, the probabilities should reflect the magnitude of the problem observed at that water surface elevation.

The actual degrees of certainties with these probabilities are highly speculative. The underseepage analyses followed in this study are indicators that piping conditions could develop. The underseepage analysis included in ETL 1110-2-556, Chapter 6 results in a gradient factor of safety of 1.0. A gradient factor of safety of 1.0 reflects a condition where floatation of particles begins and seepage and boils commence, however it is not necessarily a condition indicative of having certain levee failure. The Kansas City District has had many situations where boil activity was observed without subsequent levee failure. During the 1993 flood event on the Missouri River, the East Bottoms Unit (Station 389+54) experienced sand boils that did not result in levee failure.

Using data from the 1993 flood, the factor of safety at the initiation of boil activity was approximately 0.92.

In an effort to define a condition more representative of levee failure due to underseepage, a gradient safety factor of 0.55 was utilized in this existing conditions phase of the study. This criterion is based on the District's observation of boil activity and computed safety factors resulting from the 1952 flood on the Missouri River. An empirical relationship as shown in Table A-4.1 was developed between observed field performance and calculated factors of safety. This relationship has been effectively used by the Kansas City District since the 1960's as the basis in determining the need for underseepage treatment on levee units within the District. The effectiveness of this procedure has been demonstrated by the excellent historical performance of the District's levees in multiple flood events including the 1993 flood event on the Missouri. In the probabilistic underseepage analyses a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.86}{0.55} = 1.56 \qquad \text{Equation A-4.23}$$

where i_c is the critical gradient and FS is the gradient safety factor. The failure gradient was used to define the safety factor in Equation A-4.13 and the limit state in Equation A-4.15.

The assumptions made for the slope stability component of the risk-based analysis allowed the evaluation to be more specific as to the magnitude of the failure and the actual consequences associated with that type of failure. The slope stability analyses assumed that the failure surface should be of significant magnitude to remove the major portion of the levee allowing the interior of the levee unit to flood.

The probability of failure of the levee is also conditional on the uncertainties associated with the hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties and used in the HEC-FDA program. This is performed for economic purposes through the development of a relationship between the probability of failure of the levee and the height of water on the levees.

A-4.8.2 Soil Properties and Variations

The soil strength parameters considered in the existing conditions analysis were modeled with drained strengths for all cases. The mean values and coefficients of variations were computed from either raw data, estimated from typical values, or taken from published information.

The raw data used in this study was taken from S and R-bar effective stress tests on five projects in the Kansas City District that are considered representative of Missouri River alluvial deposits. These projects were:

L-385 is a current levee project located northwest of Kansas City, Missouri in the city of Riverside, Missouri.

The Blue River channel project is located in Kansas City, Missouri. The project extends generally southward from the mouth at the Missouri River (RM 357) upstream to 63rd Street. The Blue River is part of the East Bottoms Unit.

The Blue Springs Project is a Corps dam located on the East Little Blue tributary of the Missouri River near Blue Springs, Missouri (east side of Kansas City, Missouri).

The Longview Project is a Corps dam located on the West Little Blue River tributary in Lee's Summit, Missouri (southeast of Kansas City, Missouri).

L-142 project is a current levee project located on the left bank of the Missouri River adjacent to Jefferson City, Missouri (125 miles downstream east of Kansas City, Missouri).

The materials evaluated were designated in the Unified Soil Classification System (USCS) as CL, CH, and ML. In addition, recent soil borings throughout the Kansas City area were taken and used for soil classifications to fill in gaps with the original soil borings used in design, however no strength testing was done for this study.

The CH strength properties were calculated from tests on L-385, Blue River, Blue Springs and Longview. Through these tests, with the results shown in Table A-4.2, it was determined that the CH material had an expected value (E[$\bar{\phi}$]) of 25.3˚ with a coefficient of variation ($COV_{\bar{\phi}}$) of 15 percent. Cohesion (c) was assumed to be zero with no variation.

The CL strength properties were calculated from tests on L-385, Blue River, Blue Springs and Longview. Through these tests with the results shown in Table A-4.2, it was determined that the CL material had an expected value (E[$\bar{\phi}$]) of 29.6˚ with a $COV_{\bar{\phi}}$ of 17 percent. Cohesion (c) was assumed to be zero with no variation.

The ML strength properties were calculated from tests on L-385, Blue River, and L-142. Through these tests with the results shown in Table A-4.2, it was determined that the ML material had an expected value (E[$\bar{\phi}$]) of 29.9˚. Due to the sample size, only the mean was calculated with the raw data. Therefore, a $COV_{\bar{\phi}}$ of 11 percent was used as an approximate value with observation made by others (Kitch 1994). Cohesion (c) was assumed to be zero with no variation.

Due to lack of extensive strength data for the SM material, an expected value (E [$\bar{\phi}$]) of 32˚ was used for the material, based on limited STP blow counts correlated with relative densities and empirical correlations that were used in the L-385 design. A $COV_{\tan\bar{\phi}}$, for $\tan\bar{\phi}$, of 13.8 percent was used for the material as determined from work by others (Kitch 1994).

There was also a lack of extensive data for the strength parameters for the SP material. An expected value (E [$\bar{\phi}$]) of 34˚ was used for the material based on published data along with engineering judgment (Hunt 1984). A $COV_{\bar{\phi}}$ of 12 percent was used for this material, which was determined from work by others (Harr 1987). Strength data used compared well with recent standard penetration blow counts done on some of the units. In all cases throughout the project, the foundation sand was modeled as an SP material.

The coarse aggregate used for slope protection on the riverside slope was initially modeled as a non-random variable with an angle of internal friction of 38˚ as used on other Kansas City District projects. After several trial runs in the slope stability analyses, it was shown to have little to no affect on the analyses. Therefore, for the remainder of the analyses, the slope protection was ignored whether or not the material was present.

The embankment material in the slope was assumed to be a homogenous material having the same soil characteristics for this study. It was assumed to have a cohesion (c) of 50 psf in the slope stability analysis. The use of cohesion was solely for the prevention of unrealistic shallow sliding surfaces for the levee slopes that would create results that would not be expected. The cohesion was not considered a random variable. The expected value of the angle of internal friction (E [$\bar{\phi}$]) for the embankment material was 25° based on soil parameters used on projects L-385 and L-142 (from remolded tests). The COV $_{\bar{\phi}}$ was determined to be 16 percent as a mid-range for the CL and CH materials.

A-4.8.3 Probabilistic Underseepage Analysis Using a Flow Net

The flow net analysis was used on only one critical section that was located on the North Kansas City-Lower Unit. The analysis with a flow net was used due to the absence of any blanket material on the landward side of the levee, restricting the use of the spreadsheet hydraulic gradient procedure set forth in Kansas City District criteria.

This type of probabilistic analysis, to the best of our knowledge, had not been used before and several assumptions had to be made. In addition, procedures needed to be developed to calculate the probability of failure and/or eventually failure of the system.

The procedure involves using transformed flow net sections for three different anisotropic soil conditions. This was done in order to model the layering affect due to repeated alluvial deposits with a vertical component of permeability that is different from the horizontal component. The values assumed in the probabilistic analysis for the transformed section are listed in Table A-4.3, where the expected value is E [$k_h \backslash k_v$], the expected value at minus one standard deviation is E [$k_h \backslash k_v$] – σ , and the expected value at plus one standard deviation is E [$k_h \backslash k_v$] + σ. Example calculations for the flow net method, with the water level at the top of the levee, are available upon request.

The limit state function used in the flow net analysis was the factor of safety, which was defined as:

$$FS = \frac{i_f}{i} \qquad \text{Equation A-4.24}$$

This equation is equivalent to Equation A-4.13 where i_f is the failure gradient and i is the exit gradient. The basic probabilistic analysis was used as described in the previous sections. Example calculations of the probabilistic approach for the flow net method, with the water level at the top of the levee, are available upon request.

The analysis of the critical section discussed above was revisited in the Geotechnical Analysis North Kansas City- Lower (Harlem Area) chapter and the Geotechnical Analysis North Kansas City – Lower (National Starch Area) chapter of this appendix. The details of the reassessment can be found within those chapters, Chapter A-9 and Chapter A-10, respectively.

A-4.8.4 Probabilistic Underseepage Analysis Using the Kansas City District Criteria

The Kansas City District method of estimating the hydraulic gradients due to underseepage is slightly different than the method described in the EM 1110-2-1913. It is based on the findings made at the Missouri River Division Conference held by the Corps of Engineers in 1962 in Omaha. The underseepage analysis was based on experience during the flood event in 1952 along the Missouri River. The main differences in the Kansas City District method are:

1. The Kansas City District Method uses permeability ratios (See Table A-4.4.) related to differing material types of the blanket material in place of using actual horizontal and vertical permeabilities.

2. The Kansas City District Method assumes an infinite landside blanket in the analysis.

3. The Kansas City District Method does not use a transformed thickness for the soil stratum considered as EM 1110-2-1913 allows.

Additional information concerning the underseepage analysis for the Kansas City procedure can be found on the District's website at **http://www.nwk.usace.army.mil/local_protection/guidance.html**.

Using these underseepage analyses, the hydraulic gradient can be determined at the toe of the levee or the toe of the berm. Example calculations for determining the hydraulic gradient at the toe of the levee without a berm (with the water level at the top of the levee) are available upon request.

Critical sections were chosen based on levee height, blanket thickness, and whether or not the resulting probabilities compared well with reports made in the 1993 flood event. Results from previous studies indicated that, for underseepage, the blanket thickness was the controlling parameter. Observations made about blanket thickness in this study showed similar results.

In the probabilistic analyses of underseepage using the Kansas City District method, three random variables were considered: blanket thickness, the permeability ratio and depth of foundation sands. An exception was made at the Argentine Unit where due to a larger than average sample set of blanket thickness that were compiled, it was not considered prudent to use the blanket thickness as a random variable. Instead, the minimum blanket thickness was used. Therefore, only the permeability ratio and depth to foundation were used as random variables in conjunction with using the minimum blanket thickness of 5.5-ft at the Argentine Unit.

Since the blanket thickness was the controlling variable for all of the units, a coefficient of variation (COV_{Db}) was considered unique to each levee unit with the exception of the Argentine Unit for reasons stated above. Limited sets of borings from each unit were evaluated to determine the blanket thickness and used in the sample set to calculate the probabilistic parameters described in Section A-4.7. The probabilistic parameters were then used to determine COV_{Db} for the unit.

The depth to bedrock for the Kansas Citys - Missouri and Kansas Flood Control Project has been observed to be relatively consistent. Therefore, the COV_{Df} for depth of foundation was not varied for each levee unit and an assumed value of 6.25 percent was used (which is consistent with ETL 1110-2-556).

The $COV_{(Kf\backslash Kb)}$ of the permeability ratio was considered the same for all levee units. Using the published value given in ETL 1110-2-556, it was assumed that the $COV_{(Kf\backslash Kb)}$ was 40 percent. The permeability ratios used in the analyses followed the Kansas City District Guidance based on the type of material making up the blanket layer. In the existing conditions phase of the study the permeability ratios used in the underseepage analyses were based on material descriptions obtain from historical borings information from each unit, along with additional information from borings taken for the study within the various levee units. Table A-4.4 lists the permeability ratios.

The underseepage analyses are run using the mean values of the random variables and plus and minus one standard deviations at different river levels. Using the assumptions about the distributions and the limit state function for underseepage, a probability of failure can be developed for each river level. Example calculations for the probability of failure due to underseepage are available upon request.

A-4.8.5 Probabilistic Slope Stability Analysis

The critical section identified in the underseepage analysis was evaluated for slope stability using the model presented in ETL 1110-2-556. Several assumptions were made in order to simplify the analyses and to better represent the observation made during the 1993 flood event.

Each zone of material making up the cross sections of the levees was considered homogenous. The zones were comprised of three areas: the foundation sands, the blanket materials, and the embankment material. The foundation sands were considered constant for all levee units where the SP material was used to model the material strength. The embankment material strengths, as described in the section on material properties, were also considered constant for each levee unit. Appropriate values were assigned to the blanket material strength and properties based on the boring logs, which provided the type of blanket material (CL, CH, ML or SM).

The piezometric surface through the levee cross section was simplified and considered to be in a steady state condition. The model that was used assumed that the water surface entered the slope at the point on the riverside where the river intersected the upstream slope face. The piezometric surface then ran in a linear path to the levee toe on the landside, at the tailwater elevation.

The pore pressures developed in the blanket material were determined from the hydraulic gradient calculated at the base of the blanket material due to underseepage. The hydraulic gradient line was based on the output from the underseepage analysis using the Kansas City District Method. Assuming that the elevation head is at the same elevation as the base of the blanket material, the pore pressure (u) at a point along the base of the blanket material would be equal to the distance from the hydraulic gradient line (hp) to the base of the blanket multiplied by the unit weight of water (γ_w). The mathematical relation can be stated as follows:

$$u = h_p * \gamma_w \qquad \text{Equation A-4.25}$$

Some distance along the landward side of the levee toe, the pore pressure will dissipate through the blanket material in a linear fashion to zero at the tailwater elevation. For points within the slope, the pore pressure at the top of the blanket was calculated as the distance from the phreatic surface to the top of the blanket (h_p) multiplied by the unit weight of water (γ_w) (as in Equation A-4.25). The pore pressure at the base of the blanket was calculated using the distance from the hydraulic gradient line as the pressure head (h_p) in Equation A-4.25. A linear interpolation between these two pore pressures would give the pressure distribution through the blanket material used in the slope stability analysis.

The slope stability analyses were carried out in the same manner prescribed in ETL 1110-2-556. Utilizing the slope stability program UTEXAS 4 (for Spencer's Method), an initial circular search was performed using the expected values (means) for the random variables considered in the analysis. In order to determine a surface that would insure catastrophic failure and take out the levee, a single modal search was done at the crest of the levee. The failure surface passed through the intersection of the water surface and the slope face. Using this boundary condition, the failure would be of significant magnitude to inundate the levee interior instead of assuming a progressive slope failure from the landward levee toe.

It was also necessary to use bi-linear shear strength envelopes for the blanket and foundation materials. The bi-linear envelopes were based on the effective angles of internal friction ($\bar{\phi}$) for the material types present in the given strata of soil. The slope stability program UTEXAS 4 allows the strength of the material to be represented as numerical values. Using the Mohr-Coulomb strength criterion expressed as:

$$\tau = c + \bar{\sigma} * \tan\bar{\phi} \qquad \text{Equation A-4.26}$$

where: τ is the shear strength of the soil, c is the cohesion, $\bar{\sigma}$ is the applied normal effective stress and $\tan\bar{\phi}$ is the slope of the failure envelope on the Mohr-Coulomb strength diagram. Shear strengths for each of the material modeled in this method were developed by calculating values related to the $\bar{\phi}$ angle at its mean ($E[\bar{\phi}]$), at plus one standard deviation ($E[\bar{\phi}] + \sigma_\phi$), and at minus one standard deviation ($E[\bar{\phi}] - \sigma_\phi$) as shown in Exhibit A-4.7. Due to high pore pressures developed by hydraulic gradient from the underseepage pressure, modeling the blanket and foundation materials in this manner allowed the shear stresses to go to zero eliminating the development of negative effective shear stresses along the critical failure surfaces.

The creation of high pore pressures, due to underseepage, created unreasonably large failure surfaces that extended far beyond the landside levee toe. In order to correct for this artifact of the shear stresses going to zero in some cases, an additional boundary was set on the extent to which the failure surface could progress beyond the levee toe landward.

An initial run in the UTEXAS 4 program was made using the expected values $E[\bar{\phi}]$ for each of the different material types. The factor of safety (FS) obtained from this analysis gave the expected value for the factor of safety E [FS]. The failure surface obtained from this initial run was then considered the critical surface. The remaining series of runs were made at plus and minus one standard deviation of the expected values for strength along the critical surface defined in the initial run. As each material property was changed, a resulting factor of safety was computed. The variation resulting in each change for that particular material type can then be used in the Taylor Series Approximation. Using the probabilistic methods described previously, a probability of failure could be determined for a specific river elevation. The procedure was then repeated for various river levels and a probability curve was computed based on slope stability relationships with river levels. Example calculations for the slope stability analysis and the probabilistic analysis of the slope stability results (with the water level at the top of the levee) are available upon request.

A-4.8.6 Combined Probability of Failure Due to Slope Stability and Underseepage

The economic analysis of levee performance requires that the modal probabilities, underseepage and slope stability be combined into one curve. The levee unit must be viewed as a combined system where the total performance of the system is dependent on each mode for global stability. In evaluating the levee unit as a series system Equation A-4.11 would be expanded as follows:

$$P_{f(system)} = 1- (R_{us} * R_{ss}) = 1- [(1- P_{fus})*(1-P_{fss})] \qquad \text{Equation A-4.27}$$

where:

$P_{f(system)}$ is the probability of failure for the system
R_{us} is the reliability of the levee unit due to underseepage
R_{ss} is the reliability of the levee unit due to slope stability

A-4.9 RESULTS FOR THE RISK-BASED ANALYSES OF THE KANSAS CITYS - MISSOURI AND KANSAS FLOOD PROTECTION PROJECT

A-4.9.1 Argentine Levee Unit Results

The critical section for the Argentine Levee Unit is located at approximately Station 37+80. It is located on the upper end of the levee unit near the confluence of Barber Creek and the Kansas River. This section was chosen as the critical section due to the levee height and the thin blanket.

The typical cross section used in the analyses consisted of a 14-ft high levee with a slope of 3.5 : 1 (horizontal to vertical) on the riverside, a crest width of 15-ft, and a slope of 3 : 1 (horizontal to vertical) on the landward side.

The blanket material was determined to be an ML material with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material was approximately 5.5-ft. Due to the sample size (See Table A-4.5) used for the Argentine Unit's probabilistic parameters for blanket thickness in conjunction with the thinnest blanket in the data set the blanket thickness was not considered a random variable.

The foundation (D_f) was determined to be an SP material extending down to a depth of 62-ft.

The probabilities of failure due to underseepage, slope stability and their combined probability are shown in Exhibit A-4.8.

A-4.9.2 Armourdale Levee Unit Results

The critical section for the Armourdale Levee Unit is located at approximately Station 89+14. It is located just west of the 18^{th} Street Bridge at river mile 5.2 along the left bank of the Kansas River. This section was chosen as the critical section due to the levee height and its thin blanket.

The typical cross section used in the analyses consisted of a 13-ft high levee with a slope of 3 : 1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 3 : 1 (horizontal to vertical) on the landward side.

The blanket material was determined to be an ML material with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material was approximately 12-ft. The coefficient of variation for the blanket material (COV_{Db}) was determined as 37.7 percent from the data set shown in Table A-4.6. The foundation (D_f) was determined to be an SP material extending down to a depth of 80-ft.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.9.

A-4.9.3 Birmingham Levee Unit Results

The critical section for the Birmingham Levee Unit is located at approximately Station 200+00. It is located north of the Birmingham Wastewater Treatment Facility at approximate river mile 356 along the left bank of the Missouri River. The location chosen was a reach approximately 1200 feet long that was located between two reaches of seepage berms. According to EM 1110-2-1913 (30 Apr 2000), short reaches between two areas in which underseepage is a concern could possibly cause underseepage to concentrate in the unprotected area and recommends that the berms be extended as a continuous berm.

The typical cross section used in the analyses consisted of a 13.5-ft high levee with a slope of 3 : 1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 3.5 : 1 (horizontal to vertical) on the landward side.

The blanket material was determined to be an ML material with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material for the original design was estimated at approximately 7.0-ft which was used as the mean value for this reach. The coefficient of variation for the blanket material (COV_{Db}) was determined as 30.9 percent from very limited data taken in 1954 shown in Table A-4.7. The foundation (D_f) was determined to be an SP material extending down to a depth of 90-ft.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.10.

A-4.9.4 Central Industrial District-Kansas (CID-KS) Levee Unit Results

The critical section for the CID-KS Levee Unit is located at approximately Station 70+75. It is located about 200-ft north of the I-670 Bridge at approximate river mile 1.5 along the right bank of the Kansas River. This location was chosen for a combination of levee height and blanket thickness. Although there are areas within the data set reach which have a thinner blanket, these section had extremely low levee heights.

The typical cross section used in the analyses consisted of a 15-ft high levee with a slope of 3.5 : 1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 3.5 : 1 (horizontal to vertical) on the landward side.

The blanket material was determined to be an ML material with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material was approximately 17.5-ft. The coefficient of variation for the blanket material (COV_{Db}) was determined as 36.0 percent which was determine from the data set shown in Table A-4.8. The foundation (D_f) was determined to be an SP material extending down to a depth of 60-ft.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.11.

A-4.9.5 Central Industrial District-Missouri (CID-MO) Levee Unit Results

The critical section for the CID-MO Levee Unit is located at approximately Station 78+00. It is located about 1000-ft east of the confluence of the Missouri and Kansas Rivers at approximate river mile 267.1 along the right bank of the Missouri River.

The CID-MO Levee Unit consists mainly of floodwalls, with approximately 400-ft of levee. The typical cross section used in the analyses consisted of a 8-ft high levee with a slope of 3 : 1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 4 : 1 (horizontal to vertical) on the landward side. On the landward side of the levee, a 15-inch corrugated metal pipe was installed as an underseepage control measure.

The blanket material was determined to be an SM material with a permeability ratio (K_f/K_b) of 100. The depth of the blanket material was approximated according to its mean of 3-ft with a limited number of data points, which varied in distance from the levee centerline. The coefficient of variation for the blanket material (COV_{Db}) was determined as 62.4 percent from the data set shown in Table A-4.9. The foundation (D_f) was determined to be an SP material extending down to a depth of 70-ft.

The underseepage analysis following the Kansas City District Guidance assumed that the toe drain, as a conservative estimate, reduced the hydraulic gradient by 50 percent at the toe of the levee. During the 1993 flood event, there was no indication of underseepage problems in this area. Based on assumptions discussed previously where problems were not present at the 1993 river levels, an acceptable probability of failure was not considered to exceed 10 percent. Using the 50 percent reduction in head produced a probability of failure of approximately 1 percent.

In the slope stability portion of the analysis of this section, the hydraulic gradient's reduction due to the drainage system was ignored. The underseepage forces at the base of the blanket were considered to be under full pressure. The results indicated that there was zero probability of failure with the river level at the top of the levee.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.12.

A-4.9.6 East Bottoms Levee Unit Results

The critical section for the East Bottoms Levee Unit is located at approximately Station 389+54. It is located near the confluence of the Missouri River and the Blue River, approximately 0.5 river miles up the Blue River.

The typical cross section used in the analyses consisted of an 18-ft high levee with a slope of 3 : 1 (horizontal to vertical) on the riverside, and a stability berm that stretched 55-ft from the spring point with a slope of 20 : 1 (horizontal to vertical). The crest width was approximately 10-ft. The landward side slope was at a 4 : 1 (horizontal to vertical) with a slope stability berm running 50-ft and having a slope of 20 : 1 (horizontal to vertical). Reports from the 1993 flood event indicated that significant underseepage was observed in this area. Therefore, a probability of failure of greater than 10 percent was considered acceptable at this location for the 1993 levels.

The blanket material was determined to be a ML material, with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material was approximately 10.0-ft. The coefficient of variation for the blanket material (COV_{Db}) was determined as 60.0 percent from the data set shown in Table A-4.10. The foundation (D_f) was determined to be an SP material extending down to a depth of 70-ft.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.13.

A-4.9.7 Fairfax-Jersey Creek Unit Results

The Fairfax-Jersey Creek levee sections have extensive seepage control systems. The systems consist of wells, along with collector systems, that are maintained or replaced with regularity. It was therefore assumed that all seepage control systems are working at or above their design efficiency. It was decided to define the critical levee section as the reach of the levee where there was a complete lack of any control measures.

The critical section for the Fairfax-Jersey Creek Unit is located at approximately Station 310+00. It is located on the upper end of the levee unit north of the BPU Water and Light Plant. It lies on the right bank of the Missouri River at approximate river mile 374.

The typical cross section used in the analyses consisted of an 8-ft high levee with a slope of 4:1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 4:1 (horizontal to vertical) on the landward side.

The blanket material was determined to be an ML material with a permeability ratio (K_f/K_b) of 300. The depth of the blanket material was approximately 21-ft. The coefficient of variation for the blanket material (COV_{Db}) was determined as 36.3 percent from the data set shown in Table A-4.11. The foundation (D_f was determined to be an SP material extending down to a depth of 90-ft.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.14. It can be observed that, for the Fairfax-Jersey Creek levee system, the probability of failure is essentially zero for these geotechnical analyses.

The probability of failure related to the sheet pile wall near Station 27+50, mentioned earlier in this chapter, is discussed in Chapter A-8.

A-4.9.8 North Kansas City (Airport and Lower) Levee Units Results

During the 1993 flood event, reports indicated that the area chosen as the critical section was in a borderline failure condition. The area streets and ground (landward) in this section were reported to be heaving and pumping during the highest river level in 1993.

The critical section for the North Kansas City Levee Units is located at approximately Station 226+80. It is located south of the Harlem area of Kansas City North at approximate river mile 365.8 along the left bank of the Missouri River.

The typical cross section used in the analyses consisted of an 18-ft high levee with a slope of 3:1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a slope of 4:1 (horizontal to vertical) on the landward side.

The blanket in this section is virtually non-existent. Due to this condition, the method prescribed in the Kansas City District Guidance spreadsheet method with hydraulic gradient was not applicable. Therefore, a method using flow nets was followed for the underseepage analysis in this section. A full description of the procedure is described in Section A-4.8.

The slope stability analysis was performed using the method given in Section A-4.8. However, due to the lack of blanket, it was not necessary to interpolate the seepage pressures through the blanket for the hydraulic gradient.

The probabilities of failure due to underseepage, slope stability and the combined probability are shown in Exhibit A-4.15.

The analysis of the critical area discussed above was revisited in the Geotechnical Analysis North Kansas City- Lower (Harlem Area) chapter and the Geotechnical Analysis North Kansas City – Lower (National Starch Area) chapter of this appendix. The updated results from the reassessment can be found within those chapters.

A-4.10 SUMMARY

The geotechnical existing conditions chapter of the Kansas Citys Flood Protection Project Engineering Appendix was prepared to identify possible critical sections from a geotechnical perspective for each levee system. The probabilistic analyses performed for this study were modeled with guidance given in ETL 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" (28 May 1999).

Two modes of unsatisfactory performance were considered at various river stages- underseepage and landside slope stability under a steady state seepage condition. Riverside stability due to draw-down was not considered in this study. It was assumed that, in order for economic damages due to inundation of the interior to occur, the water level had to be higher than the landward side levee ground elevation. The stability of the riverside levee slopes during high water events are more stable, due to the additional surcharge put on the slope by the weight of the water. In addition, any destabilization due to rapid draw-down would be shallow in nature and would not result in economic damages due to interior flooding.

Where enough information was present, the probabilistic parameters needed for each of the modes were calculated. If little or no raw data was available, assumptions were made based on work done by others in the field of geotechnical risk-based analysis.

The probabilistic models used in this study were calibrated using information gained during the 1993 flood event. It was assumed that, if the levee considered showed no signs of distress at the 1993 water surface elevation, then the probability of failure should not exceed 10 percent at the 1993 water surface elevation. If significant problems were noted at the 1993 water surface elevation, the probability of failure should reflect the magnitude of the problem observed. The findings of the risk-based analyses for the eight cross sections representing the levee units considered are presented in Table A-4.12.

A-4.11 REFERENCES

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3. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Dated 1979.

4. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Appendix I, Dated 1951 - 1954.

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23. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, North Kansas City Unit Lower Section, Volume I, Dated 1978.

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A-4.12 SUPPLEMENTAL EXHIBITS AND TABLES

EXHIBIT A-4.1
Typical shape of the normal probability distribution function showing the expected value or mean, E[X]

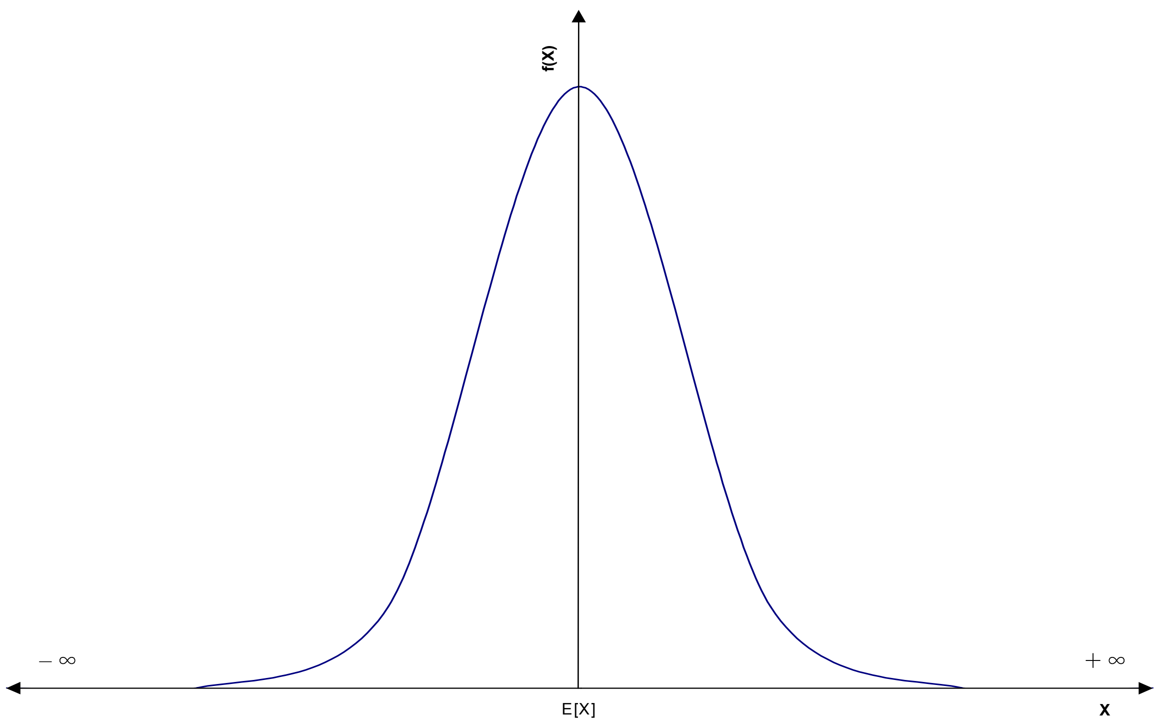

EXHIBIT A-4.2
Typical shape of the log-normal distribution function showing the expected value, E[X]

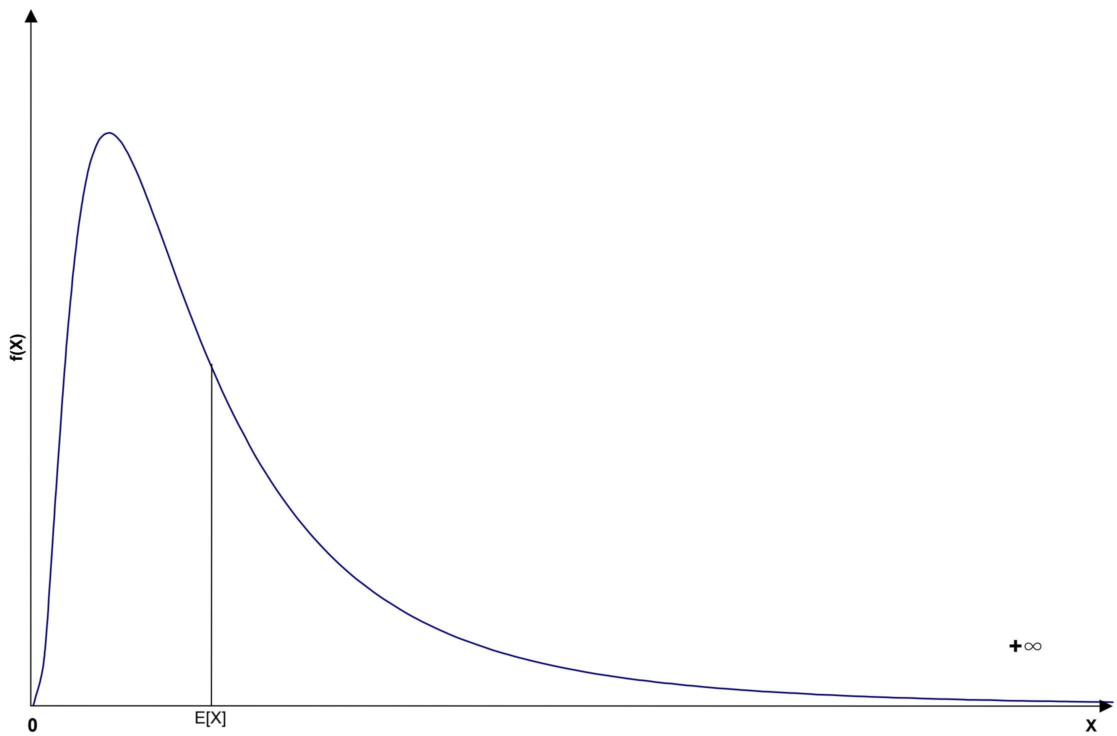

EXHIBIT A-4.3
Hypothetical normal probability distribution showing the probabilistic parameters

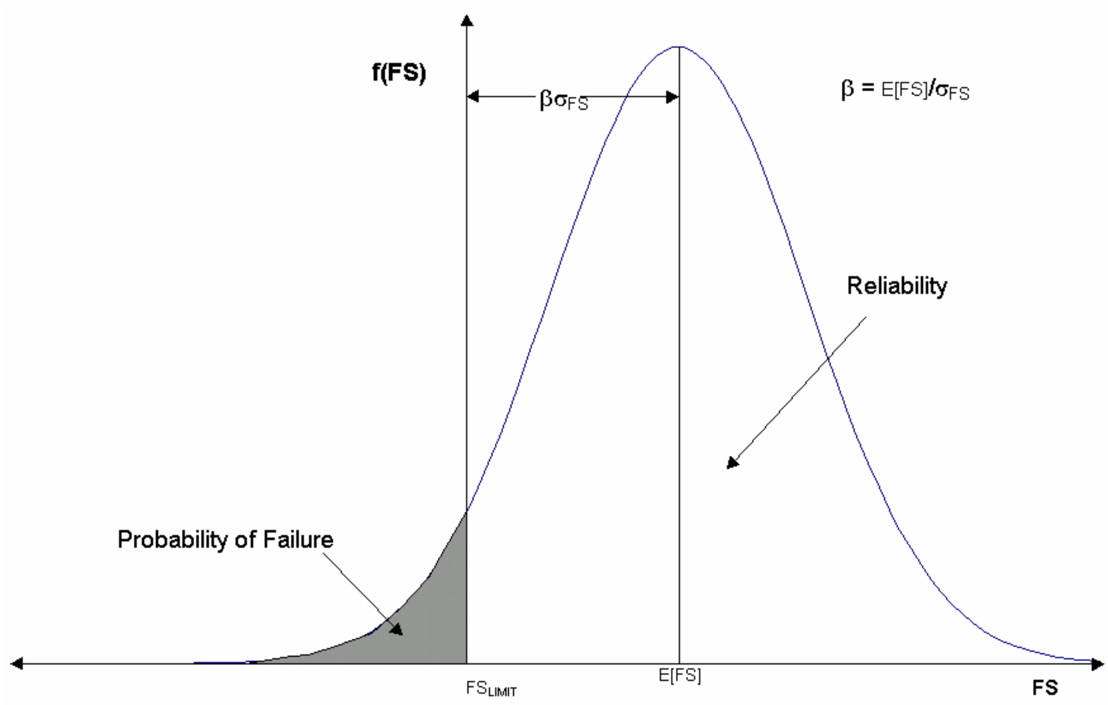

EXHIBIT A-4.4
Normal probability distribution for the natural log of the factor of safety, assuming that the factor of safety is log-normally distributed

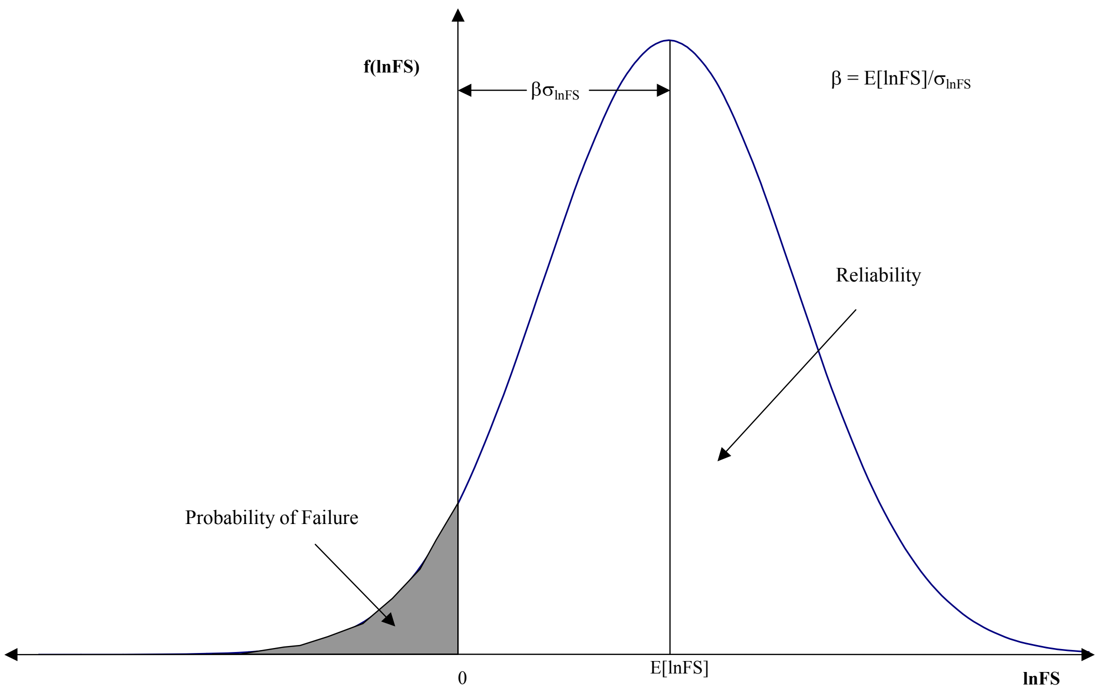

EXHIBIT A-4.5
Normal probability distribution for the natural log of the hydraulic gradient, assuming that the hydraulic gradient is log-normally distributed where the failure gradient is defining the limit state

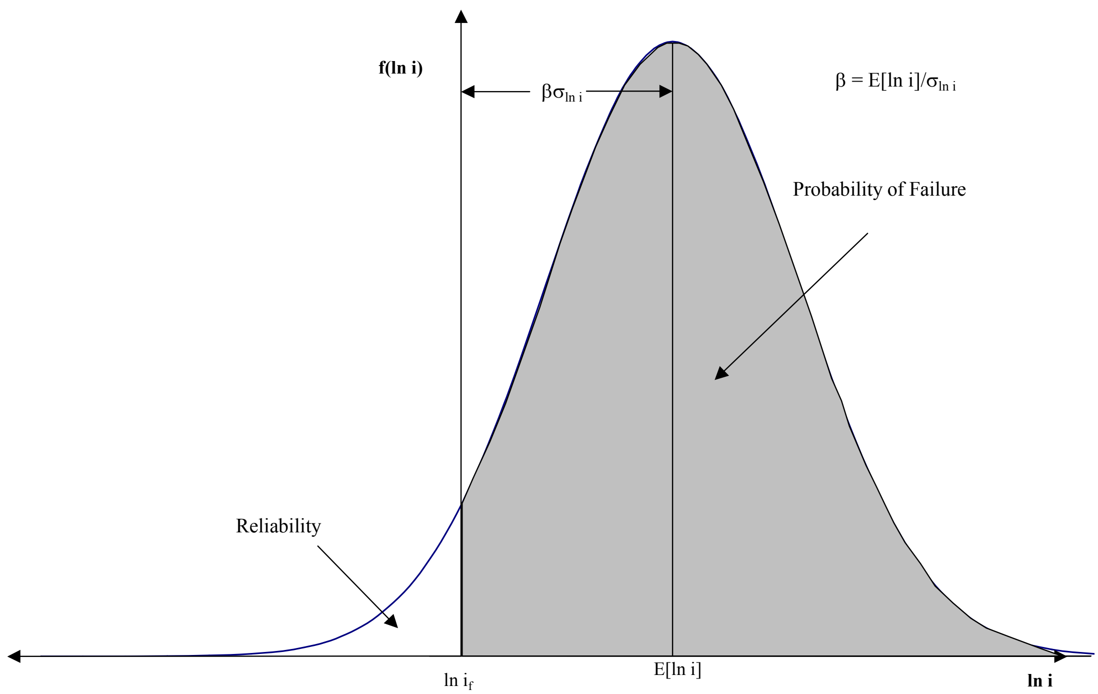

EXHIBIT A-4.6
The probability distribution curve illustrating the assumptions used in developing the Taylor Series Approximation

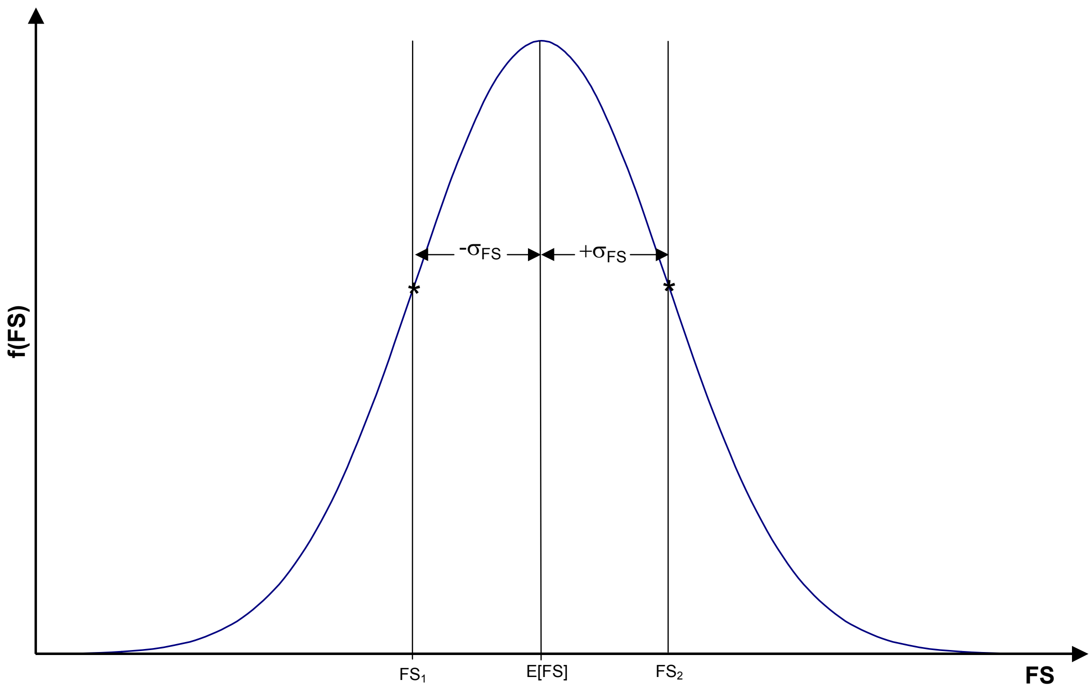

TABLE A-4.1
Observations of seepage conditions during 1952 flooding on the Missouri River at the Kansas Citys flood control project are consistent with these results

Computed Safety Factor at Flood Crest	**Seepage conditions during flood Crest**
Less than 0.55	Objectionable seepage: major flood fight; boils requiring sandbagging
0.55 to 0.80	Transition zone
Greater than 0.80	Tolerable seepage: distributed seepage, pin boils

TABLE A-4.2
Effective Strength Data Used for the CH, CL and ML Materials

Probabilitic Parameters Determined From R-bar and S Tests for Projects in the USACE Kansas City District				
S = Direct Shear R = R-bar	Location	Clasification	Angle of Internal Friction (f') deg	Material Type
S1	L-385	CH	31	CH
S2	L-385	CH	23.5	
R3	L-385	CH	30	
R4	L-385	CH	23	
R5	L-385	CH	29	
R6	L-385	CH	30	
R7	L-385	CH	25	
R8	L-385	CH	25	
R9	L-385	CH	24	
R10	L-385	CH	31	
R11	L-385	CH	28	
R12	Blue River	CH	24	
R13	Blue River	CH	25	
R14	Blue River	CH	25	
R15	Blue River	CH	25	
R16	Blue River	CH	24	
R17	Blue River	CH	24.6	
R18	Blue River	CH	22.9	
R19	Blue River	CH	20.8	
R20	Blue River	CH	15.4	
R21	Blue Springs	CH	28	
R22	Blue Springs	CH	23	
R23	Blue Springs	CH	29	
R24	Long View	CH	26.7	
R25	Long View	CH	19.2	
S1	L-385	CL	30	CL
S2	L-385	CL	21.5	
S3	L-385	CL	31.5	
S4	L-385	CL	34	
R5	L-385	CL	26	
R6	L-385	CL	28	
R7	L-385	CL	30	
R8	L-385	CL	31	
R9	L-385	CL	29	
R10	L-385	CL	34	
R11	L-385	CL	36	
R12	L-385	CL	28	
R13	L-385	CL	29	
R14	Blue River	CL	31.5	
R15	Blue River	CL	29	
R16	Blue River	CL	28	
R17	Blue River	CL	26	
R18	Blue River	CL	31	
R19	Blue Springs	CL	37	
R20	Blue Springs	CL	37	
R21	Blue Springs	CL	26	
R22	Long View	CL	30.5	
R23	Long View	CL	33	
R24	Long View	CL	27	
R25	Long View	CL	34	
R26	Long View	CL	12.5	
R1	Blue River	ML	32.5	ML
R2	Blue River	ML	31.5	
R3	L-142	ML	29	
R5	L-142	ML	26.7	

CH	MEAN	25.28
	VARIANCE	13.92
	STANDARD DEVIATION	3.73
	COEFFICIENT OF VARIATION	0.15

CL	MEAN	29.63
	VARIANCE	25.79
	STANDARD DEVIATION	5.08
	COEFFICIENT OF	0.17

ML	MEAN	29.93

TABLE A-4.3
Values Used for Flow Net Transformation on the North Kansas City-Lower Unit

$E[k_h \backslash k_v]$	$E[k_h \backslash k_v] + \sigma$	$E[k_h \backslash k_v] - \sigma$
$k_h = 3k_v$	$k_h = k_v$	$k_h = 5k_v$

TABLE A-4.4
Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permibility Ratio
SM	100
ML	200-400
ML-CL	400
CL	400-600
CH	800-1000

EXHIBIT A-4.7
Typical Bi-linear Strength Envelope

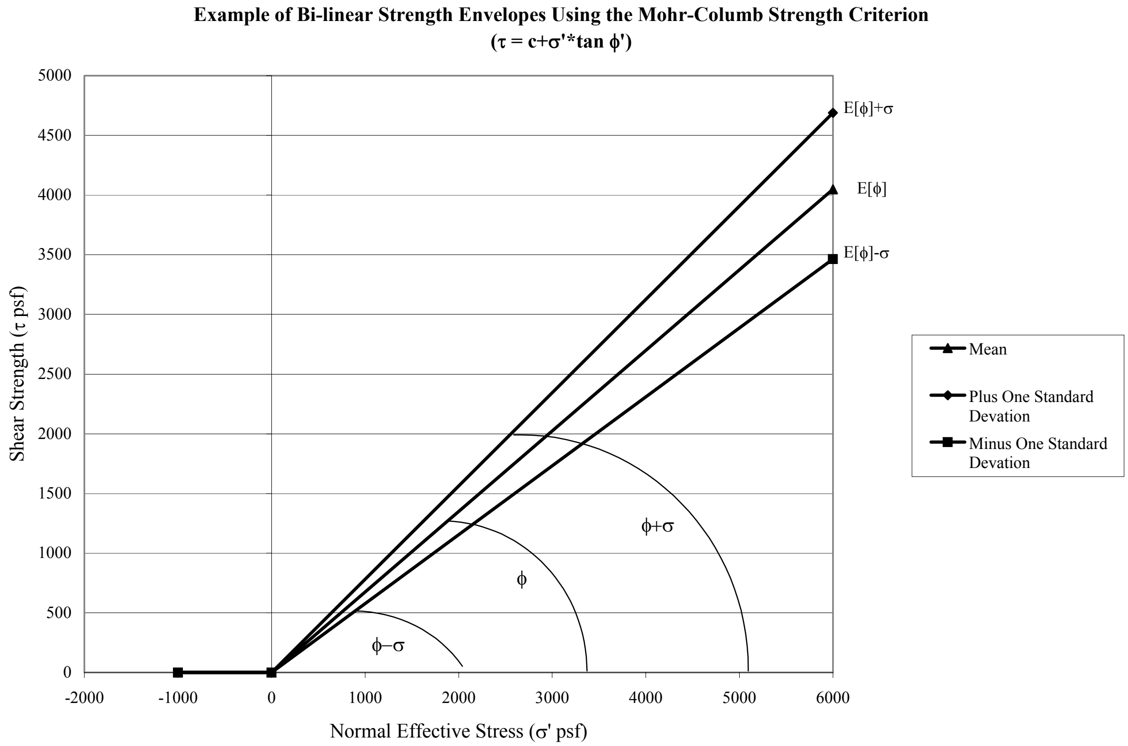

TABLE A-4.5
Probabilistic Parameters for Blanket Thickness for the Argentine Levee Unit

Boring Number	Station (ft)	Blanket Thickness (ft)	Mean Thickness (ft)	Standard Deviation (ft)	Variance (ft2)	Coefficient of Variation (%)
D-414	28+60	17.5	19.92	6.49	42.06	32.55
D-431	33+00	27.5				
D-410	34+55	36.0				
D-164	37+60	5.5				
D-108	38+80	7.5				
D-483	40+20	25.0				
D-109	42+35	18.8				
D-110	46+50	17.5				
D-38	47+58	17.5				
D-111	51+60	21.3				
D-112	55+50	16.3				
D-113	59+43	17.5				
D-477	61+10	32.5				
TP-1	62+70	15.0				
D-474	64+40	24.0				
D-473	68+00	19.0				
D-485	70+85	15.0				
D-115	73+85	7.5				
D-116	78+85	15.5				
D-117	73+05	17.5				
D-127	86+75	25.0				
D-486	90+55	25.0				
D-126	94+45	23.8				
D-471	98+00	22.5				
D-469	103+18	16.3				
D-501	105+50	23.8				
D-487	107+55	21.3				
D-488	113+28	18.8				
D-468	118+00	17.5				
D-467	118+25	13.8				
D-466	125+00	17.5				
D-489	124+95	11.3				
D-465	128+00	12.5				
D-27	132+15	12.5				
D-85	134+15	12.5				
D-128	141+55	23.8				
D-80	144+20	12.5				
D-2	148+10	20.0				
D-464	150+00	20.0				
D-26	150+85	22.5				
D-24	153+72	22.5				
D-79	164+80	17.5				
D-1	159+92	25.0				
D-492	160+18	22.5				
D-13	162+50	10.0				
D-30	165+50	11.3				
D-131	167+90	15.0				
D-493	170+20	22.5				
D-15	169+20	17.5				
D-132	172+65	16.3				
D-78	175+90	17.5				
D-84	185+55	22.5				
D-17	190+25	22.5				
D-18	200+32	22.5				
TP-6	202+80	10.0				
D-77	205+55	25.0				
D-14	210+40	28.7				
D-145	216+10	25.0				
D-81	217+50	25.0				
D-19	220+30	21.3				
D-458	221+65	6.3				
D-457	224+50	8.8				
D-41	225+00	22.5				
D-20	228+60	25.0				
D-143	229+30	22.5				
D-42	235+06	25.0				
D-144	235+30	30.0				
D-152	237+30	27.5				
D-36	240+20	25.0				
D-153	241+20	24.0				
D-496	248+20	18.8				
D-154	245+15	27.5				
D-43	245+70	27.5				
D-449	250+00	12.5				
D-446	251+40	35.0				
D-442	262+00	25.0				

EXHIBIT A-4.8
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the Argentine Unit

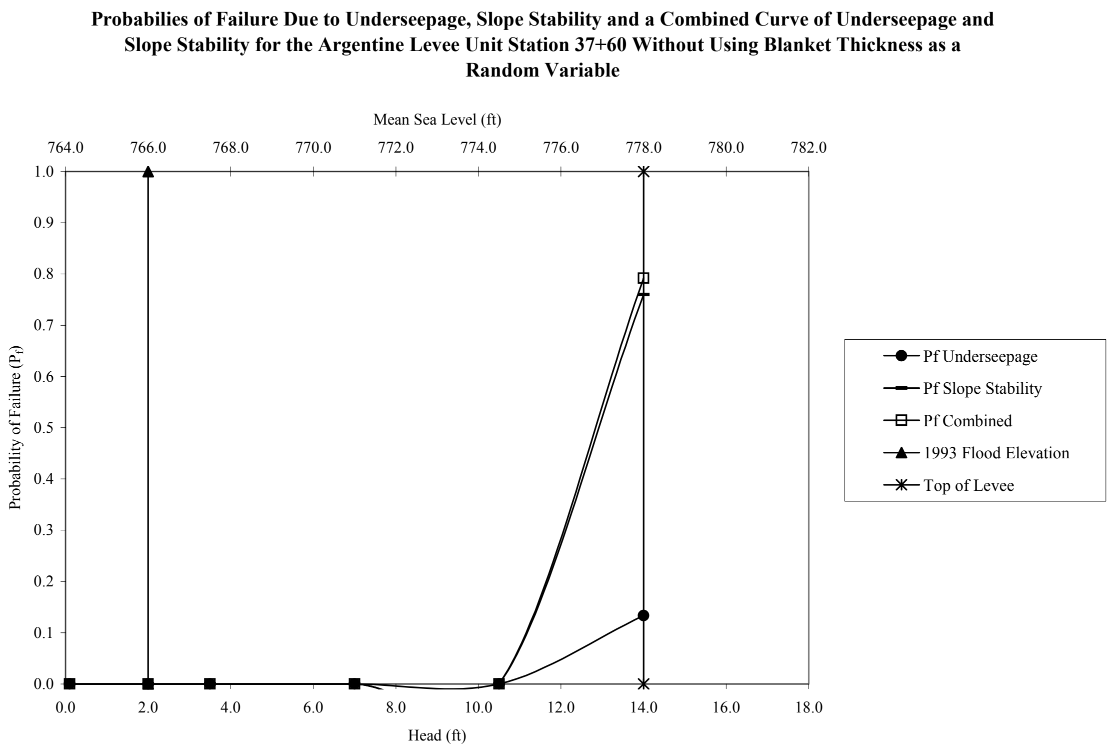

TABLE A-4.6
Armourdale Probabilistic Parameters for the Blanket Thickness
Reach 81+00 to 92+23

Boring	Armourdale Station (ft)	Blanket Thickness (ft)	Mean Blanket Thickness	Standard Devation (ft)	Coefficient of Variation (%)
D-506	81+00	20	17.7	6.7	37.7
D-349	83+18	16			
D508	86+30	30			
D-507	88+50	13			
D-356	89+14	12			
D-509	92+23	15			

EXHIBIT A-4.9
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the Armourdale Unit

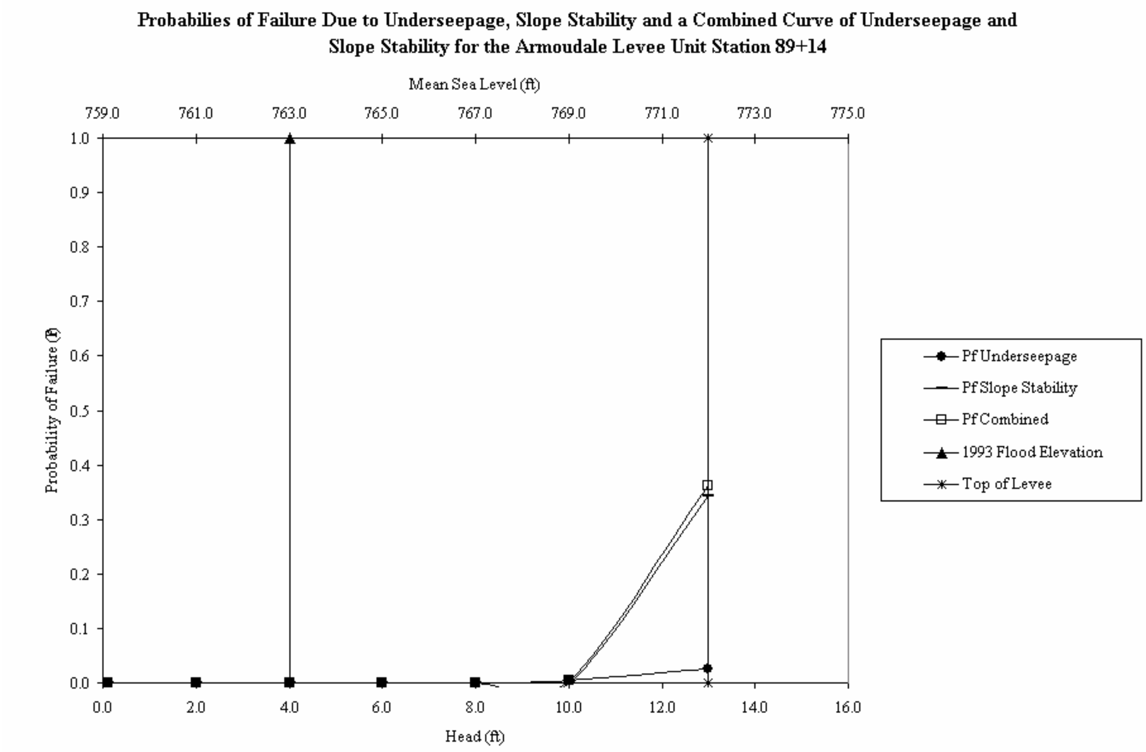

TABLE A-4.7
Birmingham Probabilistic Parameters for the Blanket Thickness Reach 189+00 to 220+00

Boring	Birmingham Station (ft)	Blanket Thickness (ft)	Mean Blanket Thickness	Standard Devation (ft)	Variance (VAR)	Coefficient of Variation
DH-82	189+00	5	4.8	1.5	2.2	30.9
DH-5	194+19	4				
DH-84	200+00	7				
DH-22	205+00	3				
DH-88	220+00	5				

EXHIBIT A-4.10
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the Birmingham Unit

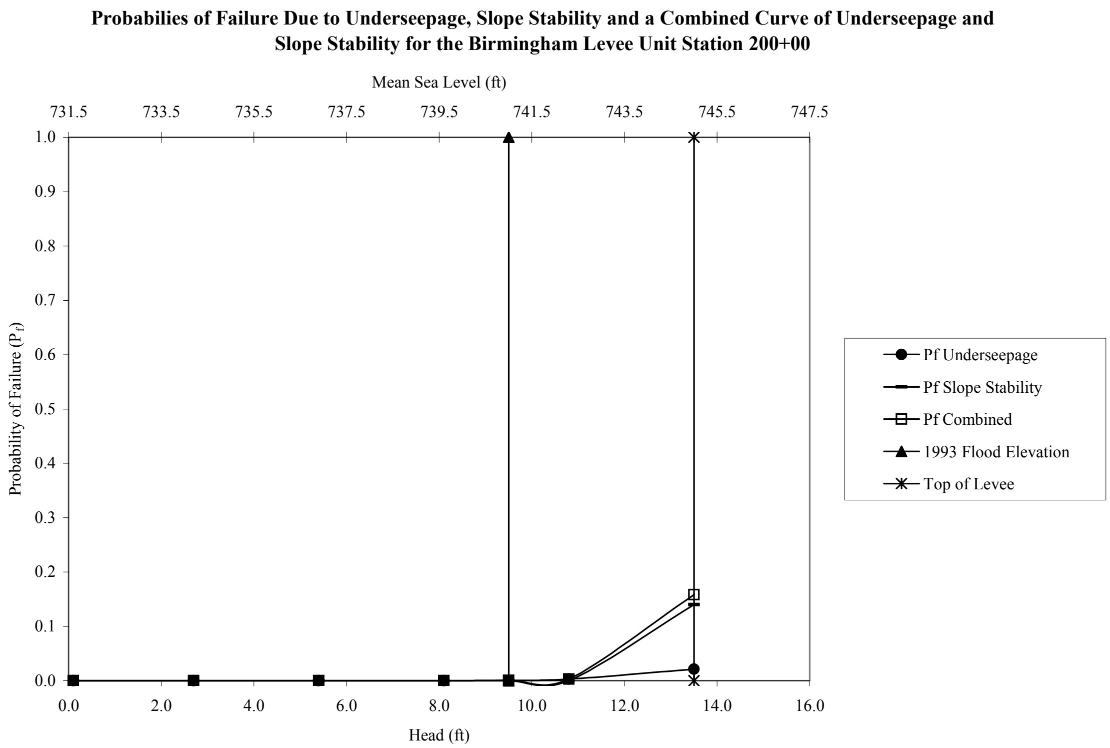

TABLE A-4.8
CID-KS Probabilistic Parameters for the Blanket Thickness
Reach 41+50 to 79+37

Boring Number	C-I-D Kansas (ft)	Blanket Thickness (ft)	Mean Thickness (ft)	Standard Deviation (ft)	Variation (ft)	Coefficient of Variation (%)
D-194	41+50	34	26.4	9.4	88.3	35.6
D-202	42+10	19				
D-191	43+45	4				
D-140	44+44	25				
D-139	44+44	40				
D-180	45+40	20				
D-135	49+00	46				
D-198	46+90	34				
D-401	46+90	30				
D-136	49+41	40				
D-372	51+39	14				
D-197	52+00	24				
D-371	52+40	17				
D-370	53+40	16				
D-133	54+42	38				
D-195	54+45	16				
D-196	54+45	18				
D-187	56+90	8				
D-201	56+75	24				
D-129	59+00	26				
D-130	59+46	36				
TP-104	60+70	14				
D-128	63+44	14				
D-127	63+44	36				
D-402	68+15	30				
W-1	68+75	28				
D-125	69+45	26				
D-349	70+42	24				
W-2	70+75	17.5				
D-403	72+00	25				
W-3	72+55	27				
W-4	73+80	26				
D-121	74+40	32				
W-5	74+85	30				
W-6	75+80	36				
D-119	76+90	31				
W-7	77+35	36				
TP-115	78+40	29				
W-8	79+30	36				
D-116	79+37	28				

EXHIBIT A-4.11
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the CID-KS Unit

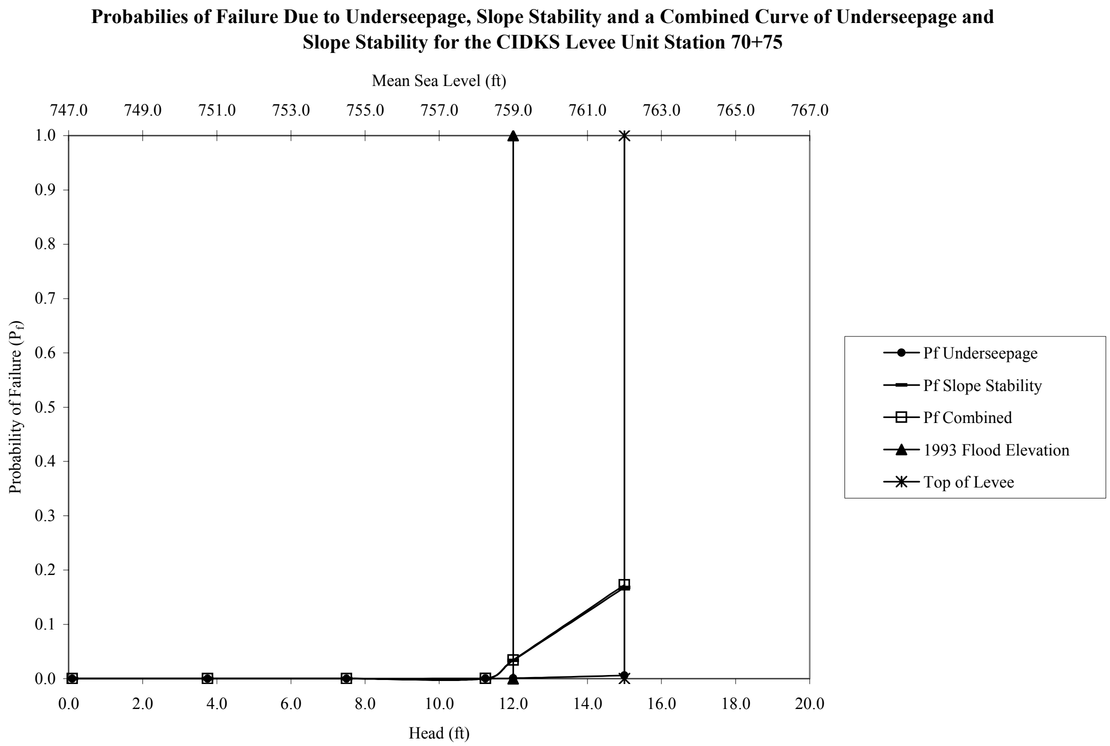

TABLE A-4.9
CID-MO Probabilistic Parameters for the Blanket Thickness Reach 78+00 to 83+00

Boring	CIDMO Station (ft)	Blanket Thickness (ft)	Mean Blanket Thickness (ft)	Standard Devation (ft)	Variance	Coefficient of Variation (%)
DH-58	75+90	5	3.0	1.9	3.5	62.4
HD-223	79+59	5				
DH-50	83+25	2				
DH-62	83+37	1				
DH-51	85+68	2				

EXHIBIT A-4.12
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the CID-MO Unit

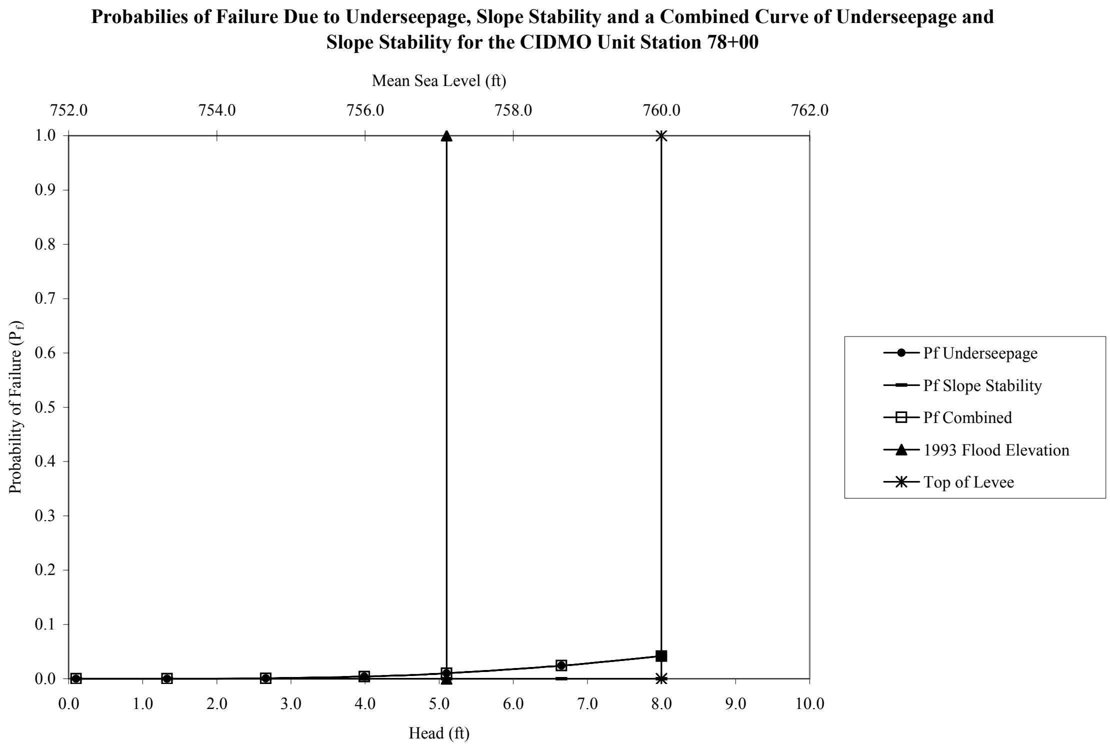

TABLE A-4.10
Probabilistic Parameters for Blanket Thickness for the East Bottoms Unit Reach 349+00 to 399+54

Boring Number	East Bottoms Station (ft)	Blanket Thickness (ft)	Mean Blanket Thickness	Standard Devation (ft)	Variance	Coefficient of Variation (%)
DH-107	349+00	7.5	12.86	7.72	59.56	60.02
DH-84	357+48	0				
DH-110	359+54	12.5				
DH-112	369+54	19				
DH-115	379+54	20				
DH-118	389+54	10				
DH-121	399+54	21				

EXHIBIT A-4.13
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the East Bottoms Unit

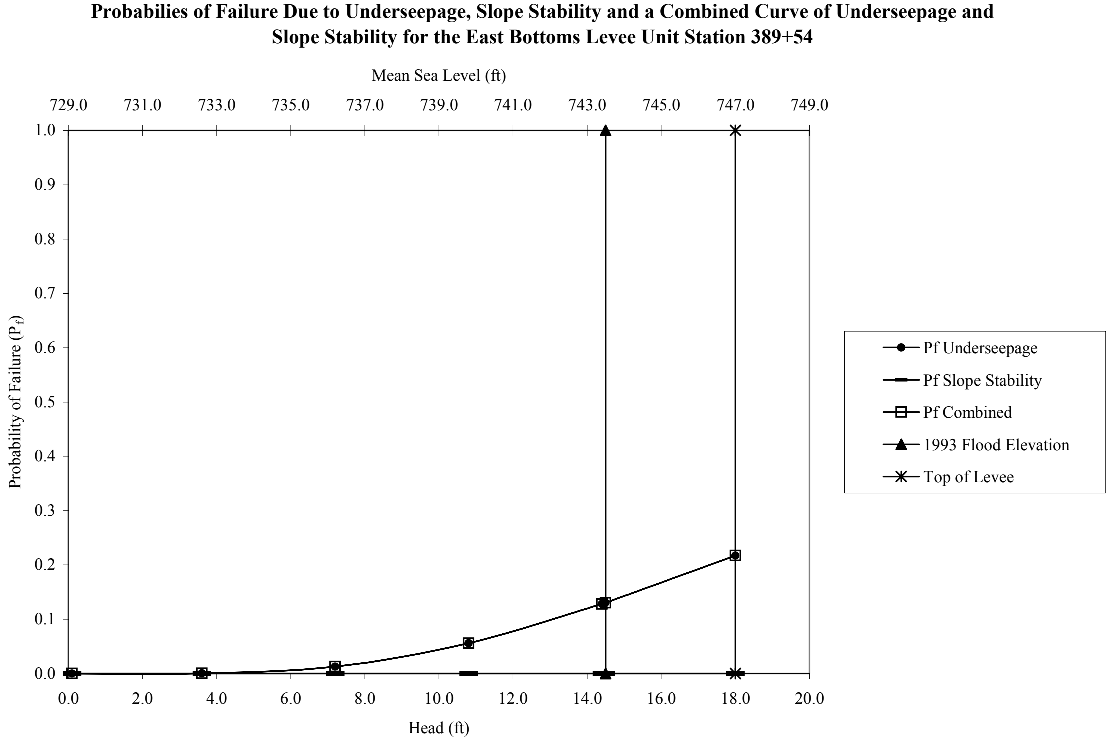

TABLE A-4.11
Probabilistic Parameters for Blanket Thickness for the Fairfax-Jersey Creek Unit Reach 302+00 to 312+25

Station (ft)	Blanket Thickness (ft)	Mean Blanket Thickness	Standard Devation (ft)	Variance	Coefficient of Variation (%)
302+00	35	26.77	9.70	94.15	36.25
303+00	35				
304+00	34				
305+00	35.5				
306+00	35				
307+00	33.5				
308+00	30				
309+00	27				
310+00	24				
311+00	21				
312+00	21				
306+00	8				
312+25	9				

EXHIBIT A-4.14
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the Fairfax-Jersey Creek Unit

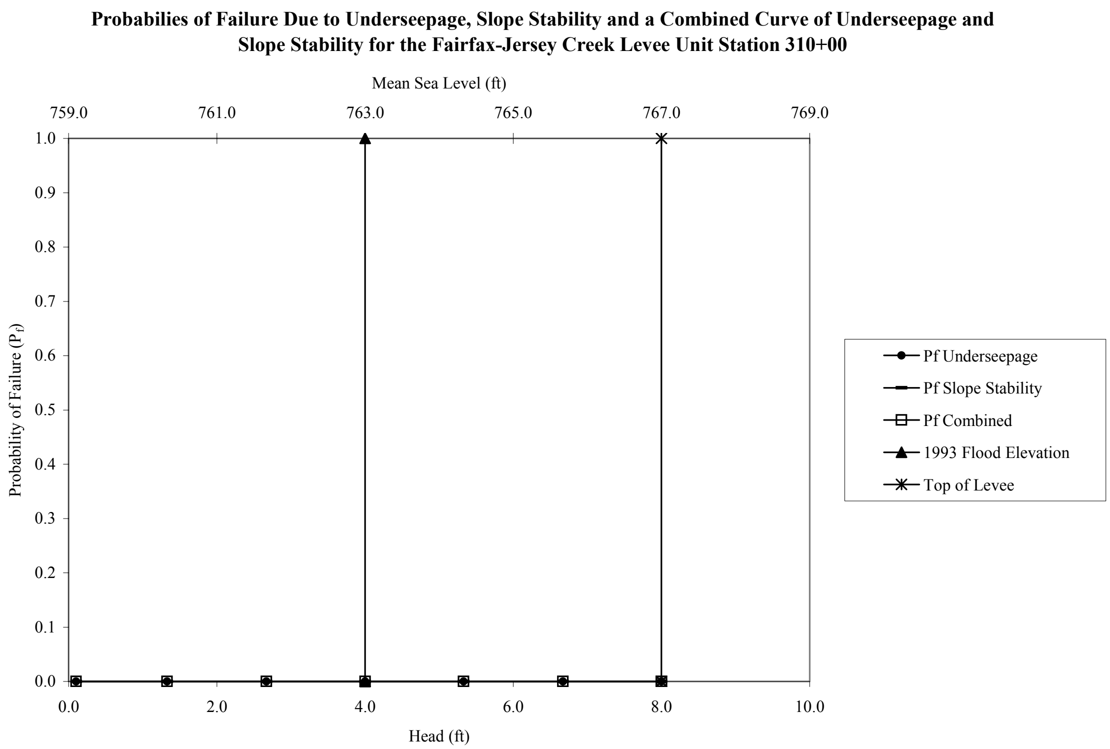

EXHIBIT A-4.15
Probability of Failure Due to Underseepage, Slope Stability and the Combined Probability for the NKC Lower Unit

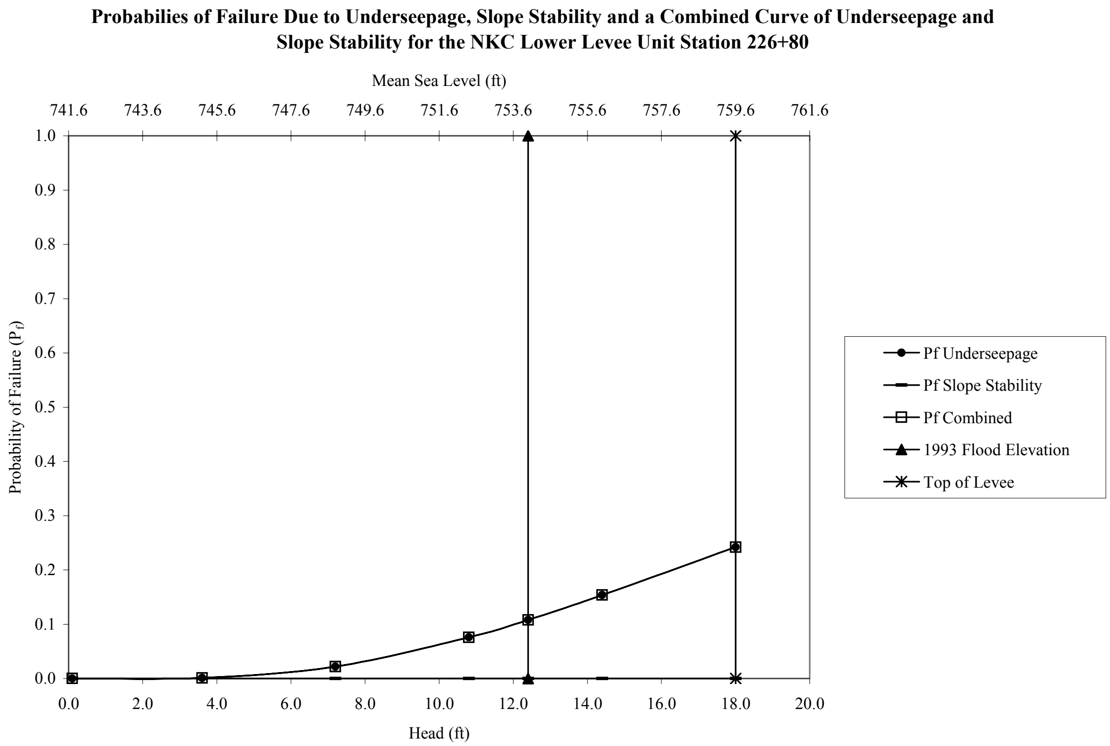

TABLE A-4.12
Summary of Risk-based Analyses of Existing Conditions for the Levee Units Considered in the Feasibility Study Giving Station and Probability of Failure (P_f) at the 1993 Flood Elevation and at the Top of Levee

Levee Unit	Station	River	P_f % (1993)	P_f % (Top of Levee)
Argentine	37+60	KS	0	79
Armourdale	89+14	KS	0	36
Birmingham	200+00	MO	0.1	16
CID-KS	70+75	KS	4	17
CID-MO	78+00	MO	1	4
East Bottoms	389+54	MO	13	22
Fairfax	310+00	MO	0	0
NKC-Lower	226+80	MO	11	24